TRANSPORTATION RESEARCH RECORD 1017

Surface Drainage and Highway Runoff Pollutants

TRB
TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL
WASHINGTON, D.C. 1985

Transportation Research Record 1017
Price $9.60
Editor: Julia Withers
Compositor: Joan G. Zubal
Layout: Theresa L. Johnson

modes
1 highway transportation
3 rail transportation

subject areas
22 hydrology and hydraulics
23 environmental design
40 maintenance

Transportation Research Board publications are available by ordering directly from TRB. They may also be obtained on a regular basis through organizational or individual affiliation with TRB; affiliates or library subscribers are eligible for substantial discounts. For further information, write to the Transportation Research Board, National Research Council, 2101 Constitution Avenue, N.W., Washington, D.C. 20418.

Printed in the United States of America

Library of Congress Cataloging in Publication Data
National Research Council. Transportation Research Board.
Surface drainage and highway runoff pollutants.

(Transportation research record; 1017)
1. Road drainage–Environmental aspects–Addresses, essays, lectures. 2. Runoff–Environmental aspects–Addresses, essays, lectures. 3. Water–Pollution–Addresses, essays, lectures.
I. National Research Council (U.S.). Transportation Research Board. II. Series.
TE7.H5 no. 1017 380.5 s 85-20513 [TD195.R63] [625.7'34]
ISBN 0-309-03906-1 ISSN 0361-1981

Sponsorship of Transportation Research Record 1017

GROUP 2–DESIGN AND CONSTRUCTION OF TRANSPORTATION FACILITIES
Robert C. Deen, University of Kentucky, chairman

General Design Section
Samuel V. Fox, Texas State Department of Highways and Public Transportation, chairman

Committee on Hydrology, Hydraulics and Water Quality
A. Mainard Wacker, Wyoming Highway Department, chairman
J. Sterling Jones, Federal Highway Administration, secretary
James E. Alleman, Frank X. Browne, Howard H. Chang, Darwin L. Christensen, Stanley R. Davis, Robert M. Engler, D. Flavell, John L. Grace, Jr., Carl M. Hirsch, Richard B. Howell, Kenneth D. Kerri, Floyd J. Laumann, Byron Nelson Lord, Walter F. Megahan, Don L. Potter, Robert E. Rallison, Brian M. Reich, J. Reichert, Everett V. Richardson, Verne R. Schneider, Robert F. Shattuck, Michael D. Smith, Charles Whittle, Ken Young, Michael E. Zeller

Lawrence F. Spaine, Transportation Research Board staff

The organizational units, officers, and members are as of December 31, 1984.

TRANSPORTATION RESEARCH BOARD
National Research Council

ERRATA 1982-1985

Special Report 201

page 17, column 2, second paragraph, should read
"Tools also need to change as the nature of options changes significantly. Emerging policy options are not largely focused on network-expansion investments, whereas traditional models were developed long ago to deal with such options."

Special Report 200

page 3, column 1
Change the caption for the bottom figure to
"A new AM General trolley bus starts down the 18 percent grade on Queen Anne Avenue North in Seattle in October 1979 (photograph by J. P. Aurelius)".

Transportation Research Record 1040

page ii
Under "Library of Congress Cataloging-in-Publication Data," delete "Meeting (64th: 1985: Washington, D.C.)" and "ISBN 0361-1981"

Transportation Research Record 1020

page 7, Figure 1
The histogram should reflect that the rail mode is represented by the black bar and that the highway mode is represented by the white bar.

Transportation Research Record 1017

page 19, column 1, 7 lines above Table 1
Change "ranged from 1 in.^{2} to nearly 30 in.^{2} of runoff" to "ranged from 1 area inch to nearly 30 area inches of runoff"

page 22, column 1, last line
Change "1 to nearly 30 in.^{2}" to "1 to nearly 30 area inches"

page 22, column 2, first line
Change "13 in.^{2}" to "13 area inches"

Transportation Research Record 1011

page 12, Figure 4
Figure does not show right-of-way structure for O-Bahn. See discussion on page 11, column 1, paragraph 3.

Transportation Research Record 996

page 49
Insert the following note to Figure 2:
"The contour lines connect points of equal candlepower."

page 49
Insert the following note to Figure 3:

"The candlepower contours are superimposed on a 'headlight's-eye-view' of a road scene. The candlepower directed at any point in the scene is given by the particular candlepower contour light that overlays that point. For example, 1400 candlepower is directed at points on the pedestrian's upper torso. For points between contour lines, it is necessary to interpolate."

page 50
Insert the following note to Figure 3:

"Where

ρ = the azimuth angle from the driver's eye to a point P on the pavement;
θ = the elevation angle from the driver's eye to a point P on the pavement;
EZ = the driver's eye height above the pavement; and
DX, DY, DZ = the longitudinal, horizontal, and vertical distance between the headlamp and eye point.

Then

$EX = EZ/\text{Tan}\,\theta \qquad HZ = EX-DZ$
$H1^2 = EZ^2 + EX^2 \qquad HX = EX-DX$
$EY = H1\ \text{Tan}\,\rho \qquad HY = EY-DY$
$H2^2 = H1^2 + EY^2 \qquad H3^2 = HX^2 + HZ^2$
$\alpha = \text{Tan}^{-1}(HZ/HX),\ \beta = \text{Tan}^{-1}(HY/H3),\ H4^2 = H3^2 + HY^2$"

Transportation Research Record 972

page 30, column 2, 22 lines up from bottom
Reference number (5) should be deleted

page 31, column 2, 5 lines up from bottom
Reference number should be 5, not 4

page 34, column 2, 8 lines above References
Reference number (5) should be deleted

Transportation Research Record 971

page 31, reference 3
Change to read as follows:

Merkblatt fürLichtsignalanlagen an Landstrassen, Ausgabe 1972. Forschungsgesellschaft für das Strassenwes en, Köln, Federal Republic of Germany, 1972.

Transportation Research Record 965

page 34, column 1, Equation 1
Change equation to

$r_u = \gamma_w \cdot h/\gamma \cdot z$

where

γ_w = unit weight of water,
γ = moist unit weight of soil,
h = piezometric head, and
z = vertical thickness of slide.

Transportation Research Record 905

page 60, column 1, 9 lines up from bottom
Change "by Payne (6)" to " by us"

Transportation Research Record 819

page 47, Table 1

Replace with the following table.

Table 1. Summary of interactions between signal-timing parameters and MOEs.

Timing Method	Parameter	Total Delay	Stops	Fuel Consumption	Emissions HC	Emissions CO	Emissions NO_x
Manual	Cycle length	⊕	⊕	⊕	⊕	⊕	⊕
	Speed of progression	+	⊕	+	+	+	+
	Priority policy	+	+	+	+	+	+
	Split method	+					
TRANSYT	Cycle length	⊕	⊕	⊕	⊕	⊕	⊕
	K-factor	+	⊕				
	Priority policy				+		

Note: + = main effect detected from TRANSYT output, and ○ = main effect detected from NETSIM output.

Transportation Research Record 869

page 54, authors' names

The second author's name should read "Edmond Chin-Ping Chang"

Transportation Research Record 847

page 50, Figure 3

Add the following numbers under each block in the last line of the flowchart:

R1, R2, R3, R4, D1, D2, D3, A1, A2

page 50, Figure 4

Make the following changes in the last line of the flowchart.

Change "R4" to "D1" and "Recognition" to "Decision"

Change "R5" to "D2" and "Recognition" to "Decision"

Change "R6" to "D3" and "Recognition" to "Decision"

Change "R7" to "R4"

Change "R8" to "D4" and "Recognition" to "Decision"

Change "R9" to "A1" and "Recognition" to "Action"

Change "R10" to "A2" and "Recognition" to "Action"

Transportation Research Record 840

page 25, column 1, line 5

Change "money" to "model"

Transportation Research Record 831

page ii, column 1

Change ISBN number to "ISBN 0-309-03308-X"

Transportation Research Circular 255

page 6, column 1, third paragraph

Change "Marquette University" to "Northern Michigan University"

NCHRP Synthesis of Highway Practice 87

page ii

Change ISBN number to 0-309-03305-5

NCHRP Synthesis of Highway Practice 84

page ii

Change ISBN number to 0-309-03273-3

TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL
2101 Constitution Avenue, N.W.
Washington, D.C. 20418

ADDRESS CORRECTION REQUESTED

Contents

Addresses of Authors

Abt, Steven R., Department of Civil Engineering, Colorado State University, Fort Collins, Colo. 80523
Bateman, J. M., Department of Civil Engineering and Environmental Sciences, University of Central Florida, Orlando, Fla. 32816
Cook, Jack S., Wilbur Smith and Associates, P.O. Box 92, Columbia, S.C. 29202
Davis, Donald R., Department of Hydrology and Water Resources, College of Engineering, University of Arizona, Tucson, Ariz. 85721
Donnell, Charles A., U.S. Army Engineering District, Charleston, S.C. 29402
Doehring, Frederick K., Department of Civil Engineering, Colorado State University, Fort Collins, Colo. 80523
Harper, H. H., Department of Civil Engineering and Environmental Sciences, University of Central Florida, Orlando, Fla. 32816
Horner, Richard R., Environmental Engineering and Science Program, FX-10, Department of Civil Engineering, University of Washington, Seattle, Wash. 98195
Howell, Richard B., California Department of Transportation, 5900 Folsom Boulevard, Sacramento, Calif. 95819
Kerri, Kenneth D., California State University, 6000 J Street, Sacramento, Calif. 95819
Mar, Brian W., Environmental Engineering and Science Program, Department of Civil Engineering, FX-10, University of Washington, Seattle, Wash. 98195
Mills, W. C., Southeast Watershed Research Laboratory, Agricultural Research Service, P.O. Box 946, Tifton, Ga. 31793
Moss, Marshall E., U.S. Geological Survey, WRD, Mail Stop 415, National Center, Reston, Va. 22092
Racin, James A., California Department of Transportation, 5900 Folsom Boulevard, Sacramento, Calif. 95819
Reich, Brian M., Brian M. Reich, Inc., 2635 Cerrada Adelita, Tucson, Ariz. 85718
Ruff, James F., Department of Civil Engineering, Colorado State University, Fort Collins, Colo. 80523
Scott, Arthur G., U.S. Geological Survey, WRD, Mail Stop 415, National Center, Reston, Va. 22092
Sheridan, Joseph M., Southeast Watershed Research Laboratory, Agricultural Research Service, P.O. Box 946, Tifton, Ga. 31793
Smith, Paul H., Wilbur Smith and Associates, P.O. Box 9412, New Haven, Conn. 06534
Thomas, Wilbert O., Jr., Environment Canada, Water Resources Branch, Inland Waters Directorate, 8th Floor, PVM Building, Ottawa, Quebec K1A OE7, Canada
Wanielista, Martin P., Department of Civil Engineering and Environmental Sciences, University of Central Florida, Orlando, Fla. 32816
Wiseman, L. P., Department of Civil Engineering and Environmental Sciences, University of Central Florida, Orlando, Fla. 32816
Yousef, Y. A., Department of Civil Engineering and Environmental Sciences, University of Central Florida, Orlando, Fla. 32816

The 1983 Santa Cruz Flood: How Should Highway Engineers Respond?

BRIAN M. REICH and DONALD R. DAVIS

ABSTRACT

Various statistical tests indicate that a significant increase in flood magnitude has occurred on Arizona's semiarid Santa Cruz River throughout its 70 years of stream gauging. Inspection of field conditions, aerial photographs, and historical reports confirms that increased conveyance and reduced overbank flooding onto wide upstream floodplains could have produced this nonstationarity. The most recent 27 years of statistical data is therefore used as a first approximation to present hydrologic conditions. Uncertainties associated with this short subrecord and progressive deterioration of the river system are considered in a systems analysis. Because of the extensive expense of rebuilding many bridges within this rapidly expanding community, it is recommended that consideration be given to confidence bands, safety factors, distributions other than the Log Pearson III, and floods observed on other Arizona watersheds within the past century in an attempt to establish a new 100-year estimate for this 2,222 square miles semiarid watershed. It is hoped that this new design parameter can be used for a reasonable planning horizon of approximately 30 years. Highway concerns include enlargement of openings for new and existing bridges, revised calculation of pier and abutment scour, general widening of the main channel that occurred during the 1983 flood, and endangerment of roadways by the river's lateral migration.

As required by the U.S. Water Resources Council, statistical analyses were performed to determine whether the hydrologic impact of progressive channel erosion of upstream floodplains during the latter part of the stream gauge record had created nonstationarity in the flood series. This led to an increased estimate of the 100-year flood (Q100); consequently, a substantiation was sought by examining watershed changes that would produce larger floods today than in the past. It also appeared prudent to examine the new Q100 in comparison to a battery of other recent estimates. These values should also be compared with the maximum floods observed on other Arizona watersheds of similar size.

EARLIER FLOODS AND ESTIMATES

Figure 1 shows 1983 damage to the Ina Road bridge that was built in 1975 with very little skew to match the channel width. The absence of enough upstream (right of the picture) bank protection led to a widening of the channel (foreground) to more than twice the length of the original bridge. Such channel widening occurred over considerable river mileage across the rapidly urbanizing Pima County. The government is about to repair or replace 12 bridges along this river as a result of the 1983 flood damage. It is important to note, however, that the damage would have been greater if significant increases in protection had not been provided by Tucson after the 1977 flood. This raises the questions of whether an even higher level of protection should be instituted for bridges and highways that might be affected, and if so, on what basis and at what level?

FIGURE 1 Ina Road Bridge of the Santa Cruz River in 1983—bank migration toward the foreground more than doubled the channel width.

A preliminary purpose of this paper is to discuss whether two major floods that occurred within the last 6 years can reasonably be accounted for through random variation in the record represented by the past 70 annual observations. The chronological series is plotted in Figure 2. An application of various statistical tests to this time series will be found later in this paper. The alternative hypothesis--that there is a progressive increase in large events--would be further supported if historical changes in the upstream river system, which

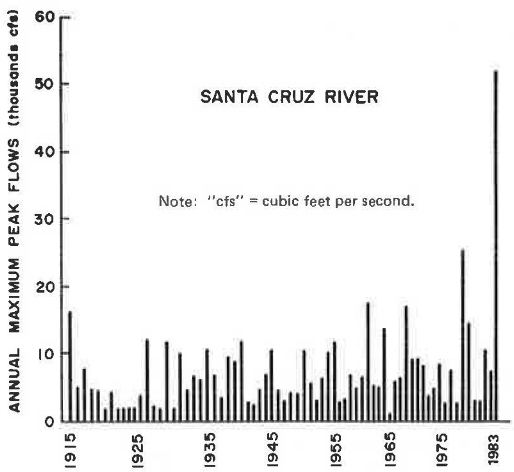


FIGURE 2 Chronological series of Santa Cruz River floods observed at Congress Street.

could produce larger flood peaks, are detected. As stated in the U.S. Water Resources Council Bulletin 17B (1,p.1) "Man's activities which can change flow conditions include urbanization, channelization, levees, construction of reservoirs, diversions, and alteration of cover conditions. . . . Special efforts should be made to identify those records which are not homogeneous." Only records that represent relatively constant watershed conditions should be used for flood frequency analysis. Inclusion of records from the earlier, more pristine watershed into a combined frequency analysis would therefore violate the need for stationarity.

The community needs to establish a scientific basis for future flood protection rather than to simply respond to each large event. The Federal Emergency Management Agency (FEMA) (2) had established the Q100 for the Santa Cruz at 30,000 ft^3/sec when the county flood maps were delivered before an event with a peak of 24,000 ft^3/sec occurred in October 1977. After the 1977 flood of record was included in an analysis of the last half of the record, it was decided to elevate the floors of a downtown redevelopment above the 45,000 ft^3/sec level. This step was appreciated after the October 1983 flood (53,000 ft^3/sec). The 1982 FEMA flood maps (3) for Tucson used 30,000 ft^3/sec based on "regional analysis including major floods from October 1977, which produced the largest peaks ever recorded at several sites" to establish the Q100 from regional regression. A 1972 U.S. Army Corps of Engineers report (4) assigned this watershed a Q100 of 46,000 ft^3/sec based on a comparison of floods observed on other rivers in southeastern Arizona.

A 1978 U.S. Geological Survey (USGS) study was performed by Roeske (5) for the Arizona Department of Transportation. The study contained analyses of Arizona data that were obtained through the 1977 water year. The regression analyses, stratified according to hydrologic regions, yielded a Q100 of 38,500 ft^3/sec for 2,200 square miles, which is the watershed area of the Santa Cruz basin above Tucson. The data did not include the large floods that occurred in southeastern Arizona in October 1977 and 1983, however. A 1984 USGS report by Eychner (6), for which major data collection was completed at the end of the 1981 water year, raised the Log Pearson Type III (LP III) Q100 station estimate of 22,100 to 23,200 ft^3/sec by weighting it with a new regional estimate. Another 1984 USGS report (7), which also did not include analysis of the October 1983 event, assigned a 40-year return period to October 1977's 24,000-ft^3/sec event. Such contradictory estimates lead to confusion and doubt among design engineers, administrators, and the public.

OBJECTIVE

The primary objective of this paper is to consolidate physical and statistical evidence that shows a change in the flood regime of this river. Because future flood-enhancing changes will probably cause a continual growth of flood peaks, there is residual uncertainty as to the proper steps to be taken. The authors believe that the flood protection level should be computed from the most recent portion of the hydrologic record because it represents the closest approximation to the present hydrologic and hydraulic conditions. Some attention should also be given to continuing the increase in mainstream and tributary conveyance upstream, which could produce larger downstream peaks in the future. This mechanism of progressive flood increase was mentioned by Zeller (8) in 1984, who assumed that non-stationarity of the hydrologic regime has developed consistently over 70 years. He refers to Q100 estimates for the Congress Street area as being 12,500 ft^3/sec in 1937 and 30,000 ft^3/sec before October 1983, and his assumed "present-day" post-October 1983 estimate is about 50,000 ft^3/sec. He states that a reliable design flood-peak estimate for a "future" 100-year flood on the Santa Cruz River (i.e., flood, or floods, occurring within the next 25 to 50 years from the present) would be 70,000 ft^3/sec because of projected man-made improvements during that period along the upstream watershed conveyance system. This is somewhat similar to multiplying his present-day estimate by a safety factor of 1.4.

The Q100 estimate by FEMA of 30,000 ft^3/sec could also be generated by multiplying the long-record pre-1983 USGS estimate (5) by a safety factor of about 1.3. This study recommends that flood-related design be based on the recognition of a changed flood regime on the Santa Cruz River. An exact description of the flood characteristics of the Santa Cruz River over the future design life of highway and bridge structures is not possible. Subsequent analysis indicates the level of this uncertainty. This uncertainty is resolved by risk analysis, that is, by consideration of the risk with and without increased levels of protection.

PHYSICAL EVIDENCE OF CHANGE

The chronological series of annual floods at Congress Street, shown in Figure 2, suggests that larger floods occurred around 1960. The population of Pima County, which is largely concentrated in and around Tucson, shows that an accelerated increase started a decade before that (Figure 3).

River Changes between Congress Street and Martinez Hill

The growth of a modern city increases the demand for sand and gravel. The dry riverbed of the Santa Cruz provided convenient sites. Laursen (9) wrote,

> I-10 from Grant Road to 29th Street is on a fill, 15 feet high, 150 feet at the ground surface and 100 feet at the top. Approximately 920,000 cubic yards went into this fill and I-19 to the south. Most, if not all, of the fill came from the river.

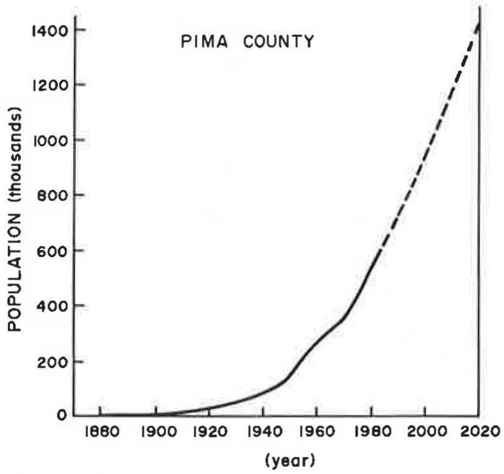


FIGURE 3 Population growth in and around Tucson.

He explained how such borrow would later progress by erosion, both upstream and downstream.

Other cultural impacts had been contributing to downcutting and widening of the Santa Cruz channel through Tucson and upstream of it since 1887. Betancourt and Turner (10) document its increases in depth and width as well as the tectonic and manmade changes from what had been high watertable, wide, vegetated floodplains with shallow, braiding sloughs. In 1960, the city discontinued incineration of garbage, and encroachments into the floodplain with sanitary landfill became common. Riparian owners and demolition contractors followed the example by toppling building debris over the river banks. It was learned through conversation with Byron Aldridge of the USGS that during these 15 years, the Santa Cruz channel bed through Tucson had dropped between 10 and 15 ft. Before this, the Work Projects Administration (WPA) had straightened several river bends between Congress Street and Martinez Hill, 7 miles upstream, which normally accelerates channel erosion.

Recent Erosion in the 7 Miles Upstream of Martinez Hill

The narrowing of the floodplain at Martinez Hill had in prehistoric times created a vegetative plug marked by 60-ft-high mesquite thorn trees. As can be seen in Figure 4, they still existed in 1940. Lowe (11) states that by 1915 the river degradation had headcut 0.5 mile past Martinez Hill, where it was controlled by a concrete dam until 1978. A 1981 photograph (Figure 5) looking upstream from Martinez Hill shows the huge conveyance that resulted from destruction of the trees. In December 1983, Betancourt (12) documented the history of this riverine environment on the San Xavier Papago Indian Reservation. Lowe described the river changes over the past 50 years throughout the 7.5 miles upstream of Martinez Hill. He traced the change over time from a shallow, small stream in a 1-mile-wide floodplain "to a deeply incised channel with more than enough capacity to carry the largest flood on record." He also discussed headcutting or deepening of tributaries in this reach. "Without the effect of these new tributary channels, overflows on the east side of the Santa Cruz would have travelled as diffuse floodplain overflow 3 to 5 miles further than they do now." Degradation of this area since 1940 progressed and, after the 1977 flood, necessitated additional riprap for the 10-year-old I-19 bridge. Six years later it was struck by about twice as much flow. Loss of one carriageway is shown in the lower right of Figure 6. In the middle distance of this aerial oblique, another downstream bridge can be seen that was outflanked by erosion of an unprotected outer bank. Another vertical aerial photograph of I-19 under construction in 1967 showed six roads from which fill was hauled from the river to

FIGURE 4 View from Martinez Hill upstream shows vegetative obstruction still accompanied a relatively small channel in 1940.

FIGURE 5 View from Martinez Hill upstream 1981, by which time destruction of the trees was complete and extensive channel widening had taken place.

FIGURE 6 Oblique aerial downstream at two bridges destroyed by bank scour immediately below Martinez Hill.

build a 2-mile elevated section paralleling the river just upstream of Martinez Hill (11). This exacerbated erosion upstream and downstream.

Changes in the Second Half of This Century Near Green Valley

Further upstream to Continental, 40 miles from Tucson, channel deepening and widening proceeded over the past 2 or 3 decades, with man's attempts to keep floods out of floodplain orchards and some fringe areas of the recently built Green Valley community. In the first half of this century, floods approaching from the south would spread out over grassy floodplains. The peak intensity that could be transmitted downstream would be attenuated by infiltration and temporary overbank storage. The 11,000-ft^3/sec, pre-1960 upper limit to Tucson floods shown in Figure 2 may have represented the throttling capacity of these many miles of natural detention areas. Today's channel efficiency speeds larger inflows from the south straight into Tucson. In fact, peaks may be further augmented by additional runoff from urbanizing downstream tributaries, which are now directly connected with the main channel.

Some validation of this hypothesis can be obtained by expressing winter Tucson flood peaks, which arise from large area storms, as a ratio to the corresponding peak at Continental, 18 miles south of Martinez Hill. Hydrographs observed at both locations in the late 1940s show that Tucson flood peaks were 0.6 times those at Continental. From 1960 through 1980, they averaged 0.85 of those entering at the upstream gauge. In October 1983, Tucson's peak was 1.5 times that estimated at Continental. This shows that upstream flood peaks are not longer reduced before they pass through Tucson.

UNDERSIZED BRIDGES AND CHANNELS

The first cost summary of repairing damage caused by the 1983 flood was published by the Pima County Department of Transportation and Flood Control District (13) only 8 days after the flood. The report covered 54 bridge sites along the five rivers that surround Tucson and included 200 before-and-after photographs, mostly taken from the air. Updated total costs for repair were estimated by Huckelberry (14) at \$20.1 million for a 55-mile stretch along the Santa Cruz. Of this total, \$13.2 million will be required for the replacement of five damaged bridges. The remaining \$6.9 million will be needed to protect seven other damaged bridges or their approaches. (Note that state bridges are not included.) Three other Tucson bridges stood because pier-reinforcing and enlarged conveyance were implemented within one year before the 1983 flood. Another city bridge that could probably withstand 100,000 ft^3/sec provided no service because the approach road was destroyed by lateral channel erosion one-half-mile upstream. This raises the issue of whether the relatively short bank protection conventionally stretching upstream and downstream of bridges is appropriate in alluvial desert situations. Baker (15) studied bank erosion after Tucson's 1983 flood and concluded:

> Where revetments remain intact during the flooding they serve to concentrate and enhance bank erosion in the unprotected reaches immediately downstream. From an overall river management perspective, piecemeal bank protection generates greater channel instability than does no protection at all.

Obviously, the \$12.2 million budgeted for bank protection in the county's flood mitigation plan falls far short of needs for the 55 miles along the Santa Cruz. Kennedy (16) of the Iowa Institute of Hydraulic Research wrote:

> Although field experience and laboratory tests have led to the establishment of fairly reliable procedures for the prediction of local scour around bridge piers, bank stability, and other such local phenomena, no such procedures exist for the analysis of alluvial riverbed and bank changes over long river reaches and extended periods of time.

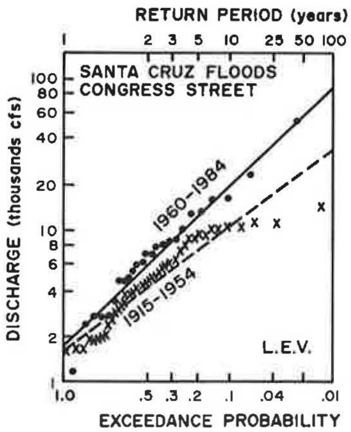
He mentions the need to incorporate the bank-erosion and channel-migration effects into models.

Probability zones for potential bank collapse at different locations behind both banks of the Rillito River, a Tucson tributary of the Santa Cruz, were developed in a recent paper by Graf (17). He summed the property values after multiplying them by their various probabilities of loss through the collapse of unprotected channel banks. This showed that the expected average annual damage due to erosion was 5 times greater than damages from flood inundation, on the assumption of fixed-bed hydraulics. This latter concept was employed by the U.S. Army Corps of Engineers to find a benefit/cost ratio of less than 1 for flood inundation along the Rillito.

If such studies were repeated with the inclusion of erosion-protection benefits, it is highly likely that federally viable projects would emerge along the Santa Cruz and its tributaries. An interesting sidelight is that in 1972, the U.S. Army Corps of Engineers (4) found a benefit/cost ratio of 1.9 to 1 for stone-revetted banks along 4 miles of the Santa Cruz between 29th Street and Grant Road. In the report it was found that the channel improvement would have been designed to control a 35-year-frequency flood of 33,000 ft^3/sec. The increased Q100, computed after 1983, as well as the assessment of collapse probabilities for land behind unprotected river banks will greatly increase benefit/cost ratios. It is hoped that this will lead to additional federal funds for the protection of these highly vulnerable areas. However, the flood frequency characteristics of the changed river regime will determine the risk involved in highway and bridge construction in the immediate future.

STATISTICAL ANALYSES AT CONGRESS STREET

One way to address the issue of whether two different periods of record were produced by the same underlying probability mechanism is to plot both records on the same probability paper. The records from 1915-1959 and 1960-1984 are plotted in Figure 7. Only the lowest value from the 1960-1984 data set overlaps the 1915-1959 data set. From the data in this figure, it can be inferred that the probability of both record segments having been generated by the same mechanism is slight. The two plots of observed floods are widely separated. The small spread around a linear trend of all but the single smallest, and thus least important, flood of the 1960-1984 series suggests that the flood series represents relatively homogeneous flood-producing watershed conditions. This linearity is not exhibited by the earlier record plotted in Figure 7, which shows that changes may have been progressing throughout the 45-year period. A number of different quantitative comparisons were made to establish the statistical reliability of the dichotomy indicated in Figure 7.



Note: "cfs" = cubic feet per second.

FIGURE 7 Graphical analog of nonparametric test to show that pre-1960 floods were distinctly smaller than more recent ones.

Means

The mean of the logarithms of the annual peak flow for the first 45 years of record is lower than that for the last 25 years of record. There is only a 5 percent chance based on the t-test that the difference could occur randomly from a single homogeneous data set.

Variances

The variance in the last 25 years of record is also higher than that in the first 45 years; random variation within a single homogeneous data set would account for the difference with a probability of 6 percent (based on an F test).

Coefficients of Skewness of the Logs

The magnitude of the flows is considered in the tests comparing the means and variances in these two data records. These tests assume that the distribution of the logarithm of the peak flows is normal. Although this may not be absolutely true, the distributions are close to normal--the sample coefficient of skew for both data sets is 0.3 ± 0.1 and the chi-square test does not reject the hypothesis that they are normal at the 5 percent level. Further, the central limit theorem would indicate that the sample statistics, which form the basis of the test, are approaching normality.

Nonparametric Tests

The Kruskal-Wallis test (18,pp.256-263) is a nonparametric test that is used to evaluate the whole series. If the two record sets are homogeneous, each record set contributes proportionately to the higher and lower values of the whole series. The record from 1960-1984 contributed proportionately more items to the higher values of the entire series, whereas the record from 1915-1959 contributed proportionately more to the lower values. There is a 7.2 percent chance that the disproportionate contributions of the two records were caused by the random variation of a homogeneous data set.

The Kruskal-Wallis test results depend only on the ranking of the peak flows, not on their magnitude. Four peak flows in the period 1960-1984 exceeded the largest flow, 15,000 ft^3/sec, in the 1915-1959 period. That the peak flow in 1984 exceeded 15,000 ft^3/sec by 38,000 ft^3/sec had no more bearing on the test results than if the exceedance had been 1 ft^3/sec. However, the advantage of the Kruskal-Wallis test is that it is independent of the type of distribution underlying the data.

Although some small uncertainty remains, the preponderance of statistical evidence indicates that there has been a change in the flood regime of the Santa Cruz River. The risk incurred by ignoring this change (15-17,19-21,pp.279-302) when determining design parameters for bridge construction is high; but how can the Q100 for the present regime be calculated?

Bulletin 17B

It is stated in Bulletin 17B (1,p.7) that "only records which represent relatively constant watershed conditions should be used for frequency analysis." The flood regime change on the Santa Cruz was not sudden. Analyses with the Kruskal-Wallis test indicate that a statistical transition occurred somewhere between 1958 and 1961. The present regime could be represented by the record encompassing the most recent 24, 25, 26, or 27 years. The calculated 100-year flood using the LP III distribution is relatively constant when data from the last 27-30 years are used. Using a shorter record increases the value of the calculated 100-year flood as the record grows shorter. The 27 yr of record from 1958 through 1984 were used with the LP III distribution and gave Q100 and Q500 (estimates for a 500-year flood) values for the Santa Cruz at Congress Street in Tucson of 49,300 ft^3/sec and 85,600 ft^3/sec, respectively.

Confidence Limits

Because flood estimates calculated here from Bulletin 17B are based on only 27 years of record, the estimates of the mean, variance, and coefficient of skew could contain considerable sample error. It is suggested in the bulletin that this uncertainty of an estimated discharge be measured by confidence limits. These confidence limits for the 100-year flood were calculated at several levels of significance and are given as follows:

Flow ft^3/sec	Chance that True Q100 Is Larger (%)
101,000	5
65,000	25
49,300	50
41,200	75
31,100	95

Note that this assigns a 50 percent chance that Q100 will be greater than 49,300 ft^3/sec. The 90 percent confidence interval extends from 31,100 to 101,000 ft^3/sec. This wide interval reflects the level of uncertainty that one could expect from one sample of a stable river regime. It is also shown there is a 5 percent chance that the Q100 exceeds 101,000 ft^3/sec.

Log Extreme Value Distribution

The point is made in Bulletin 17B that special situations may require other approaches than the Log

Pearson Type III (LP III). The Log Extreme Value (LEV) is a distribution that has been used satisfactorily in flood frequency predictions on many rivers. Figure 8, reproduced from an earlier paper by Reich (22), showed how well the LEV, fitted by the method of moments, is aligned with the 19 largest floods of the latest 25 years. By using the LEV, the Q100 for the Santa Cruz can be estimated at 96,000 ft^3/sec. The LP III curves downward to fit the 12 smaller floods. It curves 13,000 ft^3/sec below the largest flood. As a result, it can be predicted that the Q100 will be 53,000 ft^3/sec compared with 52,700 ft^3/sec, which was the maximum observed in these 25 recent years. This poor fit of the LP III curve to a larger observation results in the underprediction of the Q100. Because observed floods have smaller and random scatter about the LEV line, the indication is that this distribution may be more appropriate for the Congress Street data set.

COMPARISON WITH OBSERVED MAXIMA ON ARIZONA WATERSHEDS OF SIMILAR SIZE

Regional compatibility of a 96,000 ft^3/sec Santa Cruz estimate can be examined in Figure 9, which shows peak floods of record versus watershed area

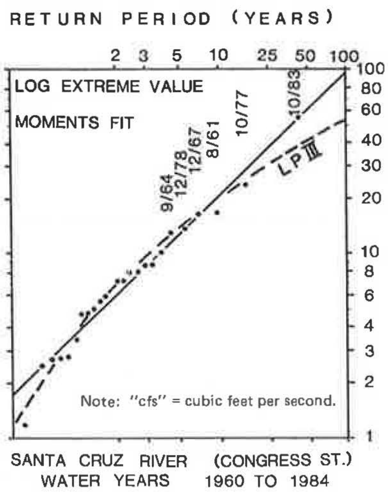


FIGURE 8 Goodness of fit of LEV straight line compared with LP-III curve for Santa Cruz.

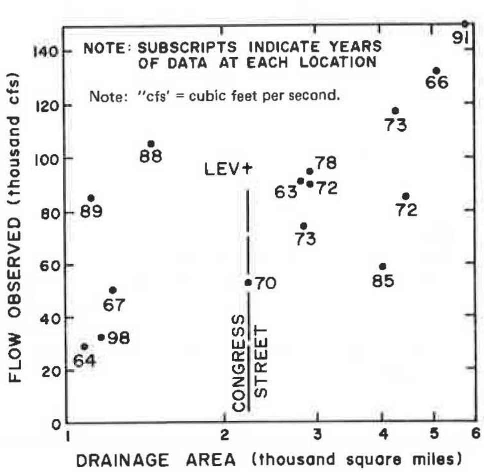


FIGURE 9 Major floods on Arizona rivers with more than 60 years of observations.

for Arizona rivers for which more than 60 years of observations were available (as indicated by the subscript at each point). The 15 data points (7,23) average 76 years of observations. On an average, one may expect the field of data points to spread around a 76-year flood trend. In fact, the Congress Street gauge did not sample present conditions for 70 years. Sparsity of storms and their random spatial location or travel directions in a semiarid climate account for some of the wide scatter in Figure 9. For instance, the 1,200-square mile San Pedro River produced 98,000 ft^3/sec in 1926 at Charleston, 70 miles southeast of Tucson. Santa Rosa Wash, 60 miles northwest of Tucson, drains 1,800 square miles and gave 53,000 ft^3/sec in 1962.

Through this Arizona data, 100-year floods should lie above the trend. For the Congress Street drainage area of 2,220 square miles, 100,000 ft^3/sec appears to be a reasonable Q100 estimate. This evidence, coupled with a probability of only 24 percent for a 100-year flood to occur in 27 years of record indicates that LP III 100-year estimate of 49,300 ft^3/sec at Congress Street may be low rather than high. Watershed changes have resulted in the Congress Street record representing less than 27 years of quasi-present flood potential. Omitting this anomalous point from Figure 9, one is left with 5 data points within the 1,700-3,500-square miles range for which bridge designs are required. No relationship with drainage area is apparent. Their observed floods ranged between 75,000 and 105,000 ft^3/sec and averaged 75 years of observation. This is an independent suggestion that Q100 estimates may exceed 90,000 ft^3/sec. This is confirmed by (a) Bulletin 17B's 5 percent confidence boundary of over 100,000 ft^3/sec and (b) the LEV estimate of Q100 at 96,000 ft^3sec.

In October 1984, Boughton and Renard (24) published an equation that enables one to estimate the maximum Q100 for any southeastern Arizona watershed from its size. It is intended to overcome the underprediction of desert floods because of poor sampling of large events in an arid region. Although their study of 18 watersheds did not include October 1983 floods, their study predicts the Q100 estimate as 77,000 ft^3/sec for the Santa Cruz in Tucson. If the study was repeated after including this record-breaking year, a modified equation may result in an estimate of approximately 100,000 ft^3/sec.

DISCUSSION

At what flood protection level should bridge and highway structures along the Santa Cruz be designed? Excessive flood protection levels result in a waste of resources. Insufficient flood protection levels increase the probability of future severe flood damage. A comparison of the risks that are involved is helpful. [See a report by Schneider and Wilson (25) for examples of the use of risk analysis in bridge design.] By viewing the reduction in expected flood damage as a benefit, and the resources required for flood protection as a cost, risk analysis provides a design wherein the benefits are greater than, or equal to the cost.

A preliminary question to be addressed is as follows: Should bridge and highway design along the Santa Cruz River be based on flood estimates obtained by considering the 70 years of record as a homogeneous record, or should design be based on the recent record representing a changed flood regime? The physical and statistical evidence for a changed regime is very strong. On a statistical basis alone, there is only a 5 to 7 percent chance that a change has not occurred. Thus, the risk of designing on the basis of an unchanged flood regime is substantially higher than the risk of design based on a changed flood regime.

The more substantive issue, however, is the level of flood protection to be called for based on the new flood regime. As indicated earlier, the short data record for the present regime causes considerable uncertainty in the calculation of the 100-year flood. Additional uncertainty is introduced by the strong possibility that the flood regime is still changing and that the potential for flooding is increasing. To completely resolve this uncertainty by using risk analysis would require detailed knowledge of construction costs and the damage to be expected because of flooding, as well as more sophisticated statistical analysis. However, the nature of the flood losses along the Santa Cruz and examination of the statistical analysis provide indications of how the uncertainty should be resolved.

In addition to overbank flood damage, considered nationally, losses from the Santa Cruz floods come from widening of the stream and changing of channel location. Losses result from undermining rather than inundation. The severity of this type of damage increases the risk associated with low levels of flood protection. Thus, economic factors indicate that uncertainty about the level of flood protection required for the Santa Cruz should be resolved by increasing the level of protection.

By using the data record from 1958 through 1984 according to the procedures in Bulletin 17B (1), a Q100 estimate at Congress Street yields 49,000 ft^3/sec. By applying confidence limit calculations, there is a 5 percent chance that the Q100 estimate might be greater than the 100,000 ft^3/sec. By using an LEV-line, the Q100 was estimated at 96,000 ft^3/sec, even without confidence limits. Interpretation of a 75-year flood from 2,222 square miles in Figure 9 would be above 80,000 cubic feet per second. None of these calculations takes into account evidence of progressive watershed change as seen by increasing channel entrenchment and growing population.

CONCLUSIONS

The following conclusions were reached:

1. The large 1983 and 1977 floods on the Santa Cruz River are indicative of the river's changed flood regime.
2. In addition to multiple statistical indications of larger flood potential on this river in the past quarter century, progressive arroyo cutting has been determined to have been a contributing factor.
3. Even without increasing the Q100 estimate for possible future increases, four independent hydrologic analyses suggest that 100,000 cubic feet per second is an appropriate Q100 estimate for the Congress Street area.

ACKNOWLEDGMENTS

The authors wish to thank the Pima County Department of Transportation and Flood Control District, and Ray Turner of the USGS for the use of their photographs.

REFERENCES

1. Guidelines for Determining Flood Flow Frequency. Bulletin 17B. Hydrology Committee, U.S. Water Resources Council, Washington, D.C., 1981, 138 pp.
2. Flood Insurance Study: Pima County, Arizona. Federal Emergency Management Agency, Dec. 1982, 211 pp.
3. Flood Insurance Study: City of Tucson, Ariz. Federal Emergency Management Agency, 1982, 45 pp.
4. Interim Report on Survey for Flood Control--Santa Cruz River Basin. Tucson, Ariz. U.S. Army Corps of Engineers, Los Angeles District, 1972, 43 pp.
5. R.H. Roeske. Methods for Estimating the Magnitude and Frequency of Floods in Arizona. Report ADOT-RS-15-121. Arizona Department of Transportation, Tucson, Ariz., 1978, 82 pp.
6. J.H. Eychner. Estimation of Magnitude and Frequency of Floods in Pima County, Arizona, With Comparisons of Alternative Methods. Water-Resources Investigations Report 84-4142. U.S. Geological Survey, 1984, 69 pp.
7. B.N. Aldridge and J.H. Eychner. Floods of October, 1977 in Southern Arizona and March 1978 in Central Arizona. Water Supply Paper 2223. U.S. Geological Survey, 1984, 143 pp.
8. M.E. Zeller. Estimating the Magnitude and Frequency of a 100-year (Design) Flood on the Santa Cruz River in the Vicinity of Tucson, Arizona from the Storm and Flood of September 28 through October 3, 1983--An Alternative Approach to Flood-Frequency Analysis. Pima County Department of Transportation and Flood Control District; Simons, Li and Associates, Inc., Tucson, Ariz., 1984, 11 pp.
9. E.M. Laursen and T. Carmody. The Santa Cruz River Above and Below Irvington Road. Ruiz Engineering Co., Tucson, Ariz., 1980, 141 pp.
10. J. Betancourt and R.M. Turner. Historical Arroyo-Cutting and Subsequent Channel Changes at Congress Street Crossing Santa Cruz River, Tucson, Arizona. Water Supply Paper. U.S. Geological Survey, Tucson, Ariz., 1985.
11. P.O. Lowe. Santa Cruz River Changes South of Tucson. Hydrology and Water Resources in Arizona and the Southwest, Vol. 14, 1984, 10 pp.
12. J.L. Betancourt. Historic Channel Changes Along the Santa Cruz River, San Xavier Reach, Southern Arizona. Terra Mar International Services, Inc., Tucson, Ariz., 1983, 33 pp.
13. Interim Flood Damage Report. Pima County Department of Transportation and Flood Control District, Tucson, Ariz., 1983, 242 pp.
14. C.H. Huckelberry. Flood Repair and Flood Mitigation Implementation Plan. Pima County Board of Supervisors, Tucson, Ariz., May 1984, 84 pp.
15. V.R. Baker. Questions Raised by the Tucson Flood of 1983. Hydrology and Water Resources in Arizona and the Southwest, Vol. 14, 1984, pp. 211-219.
16. F.M. Holly, Jr., T. Nakato, and J.F. Kennedy. Computer-Based Prediction of Alluvial Riverbed Changes. In Transportation Research Record 950, Vol. 2, TRB, National Research Council, Washington, D.C., 1984, pp. 229-237.
17. W.L. Graf. A Probabilistic Approach to the Spatial Assessment of River Channel Instability. Water Resources Research, Vol. 20, 1984, pp. 953-962.
18. W.J. Conover. Practical Nonparametric Statistics. John Wiley and Sons, Inc., New York, 1917.
19. K.D. Tsay. The Lateral Retreat of Banks In Arizona Streams. Thesis, Civil Engineering Department, University of Arizona, Tucson, 1983.
20. W.L. Graf. Variability of Sediment Removal in a Semiarid Watershed. Water Resources Research, Vol. 19, No. 3, 1983, pp. 643-652.

21. W.L. Graf. The Arroyo Problem--Paleohydrology and Paleohydraulics in the Short Term, from Background to Paleohydrology. John Wiley and Sons, New York, 1983.
22. B.M. Reich. Recent Changes in a Flood Series. Proc., Hydrology Section--Arizona-Nevada Academy of Science, Vol. 14, 1984, pp. 231-238.
23. Peak Discharge at Sites for Flood of October 1983 (Status as of July 9, 1984). U.S. Geological Survey, Tucson, Ariz., 1984, p. 1.
24. W.C. Boughton and K.G. Renard. Flood Frequency Characteristics of Some Arizona Watersheds. Water Resources Bulletin, Vol. 20, No. 5, 1984, pp. 761-770.
25. U.R. Schneider and K.V. Wilson. Hydraulic Design of Bridges with Risk Analysis. Report FHWA-TS-226, Office of Development, FHWA, U.S. Department of Transportation, 1980, 145 pp.

Cost-Effectiveness of the U.S. Geological Survey Stream-Gauging Program

MARSHALL E. MOSS, WILBERT O. THOMAS, Jr., and ARTHUR G. SCOTT

ABSTRACT

A summary of the results of a cost-effectiveness study of the U.S. Geological Survey stream-gauging program in 17 states is contained in this paper. The results are for the first year of a 5-year nationwide study undertaken by the U.S. Geological Survey. The objective of the study is to define and document the most cost-effective means of furnishing streamflow information. The first step of this study involved identification of data uses and funding sources for 1,939 continuous-record stations currently being operated with a budget of $11,425,650. Only 35 continuous-record stations were identified as not having sufficient justification to continue their operation. In addition, 31 more short-term special study stations were identified as not having justified data uses beyond completion of their respective studies. In the second step, evaluation of alternative methods of providing streamflow information, flow-routing and regression models were developed for estimating daily flows at 145 stations of the 1,939 stations analyzed. Only 6 of the 145 stations that were analyzed were considered to have acceptable accuracy of the simulated flows for the intended uses of the data. Based on the accuracy of the simulated flows, the operation of continuous-record gauging stations at these locations could be discontinued. In the third step of the analysis, relationships were developed between the accuracy of the streamflow records and the operating budget. For the current operating budget, the weighted average standard error was 21.0 percent for the programs analyzed in the 17 states. By redistribution of resources among the stations according to an optimization program, this weighted average standard error can be reduced to 19.0 percent. The current weighted average standard error of 21.0 percent can conversely be achieved with a reduced budget of $10,889,800, a total budget reduction of $535,850.

To provide basic information on the flow of the nation's streams and rivers is one of the major functions of the U.S. Geological Survey (USGS). The vast majority of this information is generated by the collection of streamflow data at some 15,000 locations throughout the United States. At approximately 8,000 of these sites, the flow of rivers, streams, or canals is continuously gauged. These gauged records are permanently stored in the Daily Values File of the USGS National Water Data Storage and Retrieval System (WATSTORE) (1). At the remaining sites, only partial records of the flow are collected. The partial-record station usually only provides data at the high (flood) or low (drought) ends of the streamflow spectrum. Many of these gauges provide the basic data required by state highway departments for the economical design of highway drainage structures. Other gauges provide data for research in rural and urban flood frequency estimation methods at gauged and ungauged sites, flow-backwater techniques, and risk analysis. In fiscal year 1983, more than $40 million was expended in the collection and processing of streamflow data by the USGS.

The first-line management of the USGS stream-gauging program is performed at the Water Resources Division (WRD) district level. WRD districts usually correspond geographically with the boundaries of one or more states. Exceptions are the Caribbean District, which includes Puerto Rico and the U.S. Virgin Islands, and the Hawaii District, which includes the Pacific Trust Territories and Hawaii.

Because of the large scale of this program and its hydrologic and managerial complexities, a considerable effort has been expended within the National Research Program of the WRD to develop technologies for the design and management of data collection programs. As new tools became operational, they were readily implemented in the district programs. The nationwide implementation of one such set of tools is described by Benson and Carter (2). Subsequent to this nationwide study in the early 1970s, no new technologies were developed that had sufficient impact on the program to warrant another nationwide study until 1980. In 1980, Moss and Gilroy (3) presented a new approach to measuring the cost-effectiveness of a stream-gauging program. By using this technique, called the Kalman-Filtering for Cost-Effective Resource Allocation (K-CERA), the manager of a stream-gauging program can evaluate allocations of gauging effort among the continuous stream gauges of the program such that the overall amount of information that is generated would be a maximum. The K-CERA is composed of a set of techniques and computer programs to estimate measures of the errors in streamflow estimates and to distribute fiscal resources in a network to minimize the sum of error variances of each site, which, in turn, maximizes information. However, the approach does not specify the set of gauges that should make up the program. To address this last point, other steps are required.

The potential impact of the K-CERA as a management tool led the USGS to initiate another nationwide analysis of its stream-gauging program in 1982 with this approach as its basis. Because of the relatively large initial investment of manpower in the implementation of the K-CERA, it was decided that the nationwide study would be performed over a 5 year period. In each of the 5 years, managerial units, usually districts, that contained 20 percent of the stream-gauging program would complete the analyses. Locations and areal extents of the studies performed during the first year are shown by the shaded areas in Figure 1. During 1983, the stream-gauging program was analyzed in Alaska, Arkansas, northern California, central Florida, Georgia, Hawaii (including the Pacific Trust Territories), Idaho, Illinois, Iowa, Kansas, Maine, Massachusetts, Nebraska, New Jersey, Pennsylvania, Rhode Island, and Washington. These analyses are summarized in this report for the first study year.

FIGURE 1 Locations and areal extents of the studies performed during the 1983 fiscal year.

APPROACH

An analysis of a stream-gauging program would, ideally, define the proper set of stream gauges to be operated and specify the most cost-effective way to operate those gauges. The K-CERA addresses the second aspect, but no robust technology for definition of the proper set of stream gauges currently exists. A pragmatic approach that consists of three sequential steps is therefore being used to analyze each of the continuous stream gauges in the USGS stream-gauging program. The first two steps involve screening each gauge in the program as to whether it should remain in use as a continuous stream gauge. In the first step, all known uses of the data that are generated at each continuous-record site are documented for comparison with other data collected as part of the USGS mission of generating streamflow information. Those stream gauges with uses that are not found to be sufficient and compatible with the USGS mission are suggested for discontinuance. Additionally, funding for the operation of each gauge and the frequency of the availability of its data are also documented.

In the second step, those gauges that passed the first screening are investigated as to whether a sufficient amount of the streamflow information contained in the streamflow data can be generated by means of either hydrologic models or statistical methods. These alternative methods of generating streamflow information are less costly than operating a continuous stream gauge. No guidelines concerning suitable accuracies exist for particular uses of the data. Therefore, judgment is required in deciding whether the accuracy of the estimated daily flows is suitable for the intended purpose. If the alternative method is successful for a particular stream gauge, then that stream gauge becomes a candidate for discontinuance.

Those gauges that pass the first two steps make up the continuous stream-gauging program that is to be subjected to the K-CERA analysis for the determination of its optimal operation in terms of cost-effectiveness.

A brief description of the content of each of these steps follows. However, if more details are desired, see the report by Fontaine et al. (4), which served as a prototype for all of the other areas analyzed in 1984.

STEP ONE--CATEGORIZATION BY DATA USE, FUNDING, AND FREQUENCY OF AVAILABILITY

Data Use Categories

The following definitions were used to categorize each known use of streamflow data for each continuous stream gauge. A given station may be included in more than one data use category.

Regional Hydrology

For data to be useful in defining regional hydrology, a stream gauge must be largely unaffected by man-made storage or diversion. In this category of uses, the effects of man on streamflow are not necessarily small, but the effects are limited to those caused primarily by land use and climate changes. Large amounts of man-made storage may exist in the basin, providing that the outflow is uncontrolled. These stations are useful in developing regionally transferable information about the relationship between basin characteristics and streamflow.

Hydrologic Systems

Stations that can be used for accounting (i.e., for defining current hydrologic conditions and the sources, sinks, and fluxes of water through hydrologic systems that include regulated systems) are designated as hydrologic systems stations. They include diversions and return flows and stations that are useful for defining the interaction of water systems.

The benchmark and index stations are included in the hydrologic systems category because they account for the current and long-term conditions of the hydrologic systems that they gauge. Federal Energy Regulatory Commission (FERC) stations and international gauging stations, located on significant rivers that cross national boundaries, are also included.

Legal Obligations

Some stations provide records of flows for the verification of enforcement of existing treaties, compacts, and decrees. The legal obligation category contains only those stations that the USGS is required to operate to satisfy a legal responsibility.

Planning and Design

Gauging stations in this category of data use are used for the planning and design of a specific project (for example, a dam, levee, floodwall, navigation system, water-supply diversion, hydropower plant, or waste-treatment facility) or group of structures. The planning and design category is limited to those stations where these purposes are currently valid.

Project Operation

Gauging stations in this category are used, on on ongoing basis, to assist water managers in making operational decisions such as reservoir releases, hydropower operations, or diversions. The project operation use generally implies that the data are routinely available to the operators on a rapid-reporting basis. For projects on large streams, data may only be needed every few days.

Hydrologic Forecasts

Gauging stations in this category are regularly used to provide information for hydrologic forecasting. These might be flood forecasts for a specific river reach, or periodic (daily, weekly, monthly, or seasonal) flow-volume forecasts for a specific site or region. The hydrologic forecast use generally implies that the data are routinely available to the forecasters on a rapid reporting basis. On large streams, data may only be needed every few days. Data used for forecasting inflows or outflows solely for project operation are categorized as project operation and are not contained in the forecast category.

Water-Quality Monitoring

Gauging stations where regular water-quality or sediment-transport monitoring is being conducted and where the availability of streamflow data contributes to the utility or is essential to the interpretation of the water-quality or sediment data are designated as water-quality-monitoring sites.

Research

Gauging stations in the research category are operated for a particular research or water-investigation study. Typically, these are only operated for a few years.

Other

The eight categories described previously contain the majority of data uses. However, occasional data uses have been identified that do not fit into the scheme. Therefore, the "other" category is provided for such instances.

Funding

The four funding sources for the streamflow-data program are as follows:

1. Federal--Funds that have been directly allocated to the USGS.
2. Other federal agency (OFA)--Funds that have been transferred to the USGS by OFAs.
3. Cooperative (COOP)--Funds that come jointly from USGS cooperative-designated funding and from a nonfederal cooperating agency. Cooperating agency funds may be in the form of direct services or cash.
4. Other nonfederal--Funds that are provided entirely by a nonfederal agency or a private concern under the auspices of a federal agency. In this study, funding from private concerns was limited to that derived from the licensing and permitting requirements for hydropower development by the FERC. Funds in this category are not matched by USGS cooperative funds.

In all four categories, the identified funding sources pertain only to the collection of streamflow data. Funding sources for other activities, particularly collection of water-quality data, which might be carried out at the site, may not necessarily be the same as those identified herein.

Frequency of Data Availability

Frequency of data availability refers to the times at which the streamflow data may be furnished to the users. In this category, three distinct possibilities exist. Data can be furnished by (a) direct-access telemetry equipment for immediate use, (b) periodic release of provisional data, or (c) in the annual data reports published by the USGS.

STEP TWO--CONSIDERATION OF ALTERNATIVE METHODS

Two methods were used to synthesize streamflow records at gauging stations where it was thought that the records were sufficiently correlated with the records of one or more other stations. These two methods are described briefly in the following. Usually no more than 10 percent of the gauges in any district program were candidates for the alternative-methods analysis.

Description of Flow-Routing Model

Hydrologic flow-routing methods use the law of conservation of mass and the relationship between the storage in, and outflow from, a reach. The hydraulics of the system are not considered. The method usually requires only a few parameters and treats the reach in a lumped sense without subdivision. The input is usually a discharge hydrograph at the upstream end of the reach and the output, a discharge hydrograph at the downstream end. Several different types of hydrologic routing are available such as Muskingum, Modified Puls, Kinematic Wave, and the unit-response flow-routing method. The last method was selected for this analysis. Two techniques are used--storage continuity (5) and diffusion analogy (6,7). The computer program that utilizes these two techniques of flow routing is described by Doyle et al. (8).

Description of Regression Analysis

Simple and multiple-regression techniques were also used to estimate daily flow records. Regression equations relate daily flows at a single gauge to daily flows at a combination of upstream, downstream, and (or) tributary gauges. This statistical method is not limited, as is the flow-routing method, to gauges where an upstream gauge exists on the same stream. The explanatory variables in the regression analysis can be data from gauges in different watersheds or in downstream and tributary watersheds. The regression method has many of the same attributes as the flow-routing method in that it is easy to apply, provides indexes of accuracy, and is generally accepted as a good tool for estimation. The theory and assumptions of regression analysis are described in several textbooks such as that by Draper and Smith (9) and that by Kleinbaum and Kupper (10). The application of regression analysis to hydrologic problems is described and illustrated by Riggs (11).

STEP THREE--K-CERA

In a study of the cost-effectiveness of a network of stream gauges that was operated to determine the amount of water consumption in the Lower Colorado River Basin, a set of techniques called the K-CERA was developed (3). Because of the water-balance nature of that study, the measure of effectiveness of the network was chosen to be the minimization of the sum of variances of errors of estimation of annual mean discharges at each site in the network. This measure of effectiveness tends to concentrate stream-gauging resources on the larger, less stable streams where potential errors are greatest. Although such a tendency is appropriate for a water-balance network, in the broader context of the multitude of uses of the streamflow data collected in the USGS's Streamflow Information Program, this tendency causes undue concentration on larger streams. Therefore, the original version of the K-CERA was extended to include, as optional measures of effectiveness, the sums of the variances of the following: errors of annual mean discharge estimation in cubic feet per second and percent, and errors of average instantaneous discharge estimation in cubic feet per second and percent. The use of percentage errors does not, however, unduly weight activities at large streams to the detriment of records on small streams. In addition, the instantaneous discharge is the basic variable from which all other streamflow data are derived. For these reasons, this study used the K-CERA approach with the sums of the percentage error variances of the instantaneous discharges at all continuously gauged sites as the measure of the effectiveness of the data-collection activity.

Brief descriptions of the mathematical program that was used to optimize cost-effectiveness of the data-collection activity and of the application of Kalman filtering (12) to the determination of the accuracy of a stream-gauging record are presented in the following paragraphs. For more detail on the theory, the assumptions, or the applications of the K-CERA, see reports by Moss and Gilroy (3), Gilroy and Moss (13), and Fontaine et al (4).

Description of Mathematical Program

One program in the K-CERA technique is called "the traveling hydrographer." This program attempts to allocate among stream gauges a predefined budget for the collection of streamflow data so that the field operation is the most cost-effective possible. The measure of effectiveness was discussed previously in this paper. The set of decisions available to the manager is the use frequency (number of times per year) of each of a number of routes that may be used to service the stream gauges and to make discharge measurements. The range of options within the program is from zero to daily usage for each route. (A route is defined as a set of one or more stream gauges and the least cost travel that takes the hydrographer from his base of operations to each of the gauges and back to base.) A route will be associated with an average cost of travel and an average cost of servicing each stream gauge visited along the way. The first step taken by the analyst is to define the set of practical routes. This set of routes will frequently contain the path to an individual stream gauge with that gauge as the lone stop and return to the home base so that the individual needs of a stream gauge can be considered in isolation from the other gauges.

The analyst then determines any special requirements for visits to each of the gauges for such things as necessary periodic maintenance, servicing of recording equipment, or required periodic sampling of water-quality data. Such special requirements are considered to be inviolable constraints in terms of the minimum number of visits to each gauge.

The final step is to use the traveling hydrographer program with all of the above to determine the number of times that the routes are used during a year such that (a) the budget for the network is not exceeded, (b) the minimum number of visits to each station is made, and (c) the total uncertainty in the network is minimized.

Description of Uncertainty Functions

As noted earlier, uncertainty in streamflow records is measured in this study as the average relative variance of estimation of instantaneous discharges. The accuracy of a streamflow estimate depends on how that estimate was obtained. Three situations are considered in this study:

1. Streamflow is estimated from measured discharge and correlative data using a stage-discharge relation (rating curve),
2. The streamflow record is reconstructed using secondary data at nearby stations because primary correlative data are missing, and
3. Primary and secondary data are unavailable for estimating streamflow.

The variances of the errors of the estimates of flow that would be employed in each situation were weighted by the fraction of time each situation is expected to occur. The average relative variance would thus be

$$\bar{V} = \epsilon_f V_f + \epsilon_r V_r + \epsilon_e V_e \quad (1a)$$

with

$$1 = \epsilon_f + \epsilon_r + \epsilon_e \quad (1b)$$

where

$\bar{V}$ = the average relative variance of the errors of streamflow estimates,
ε_f = the fraction of time that the primary recorders are functioning,
V_f = the relative variance of the errors of flow estimates from primary recorders,
ε_r = the fraction of time that secondary data are available to reconstruct streamflow records given that the primary data are missing,
V_r = the relative variance of the errors of estimation of flows reconstructed from secondary data,
ε_e = the fraction of time that primary and secondary data are not available to compute streamflow records, and
V_e = the relative error variance when both primary and secondary data are not available.

The fractions of time that each source of error is relevant are functions of the frequencies at which the recording equipment is serviced.

The relative variance, V_f, of the error derived from primary record computation is determined by analyzing a time series of residuals that are the differences between the logarithms of measured discharge and the rating curve discharge. The rating curve discharge is determined from a relationship between discharge and some correlative data, such as water-surface elevation at the gauging station. The measured discharge is the discharge determined by field observations of depths, widths, and velocities.

If the recorder at the primary site fails and there are no concurrent data at other sites that can be used to reconstruct the missing record at the primary site, there are at least two ways of estimating discharges at the primary site: a recession curve could be applied from the time of recorder stoppage until the gauge was again functioning, or the expected value of discharge for the period of missing data could be used as an estimate.

The expected-value approach is used in this study to estimate V_e, the relative error variance during periods of no concurrent data at nearby stations. If the expected value is used to estimate discharge, the value that is used should be the expected value of discharge at the time of year of the missing record because of the seasonality of the streamflow processes. The variance of streamflow, which also is a seasonally varying parameter, is an estimate of the error variance that results from using the expected value as an estimate.

The variance V_r of the relative error during periods of reconstructed streamflow records is estimated on the basis of correlation between records at the primary site and records from other gauged nearby sites.

Because errors in streamflow estimates arise from three different sources with widely varying precision, the resultant distribution of those errors may differ significantly from a normal or log-normal distribution. This lack of normality causes difficulty in interpretation of the resulting average estimation variance. When primary and secondary data are unavailable, the relative error variance V_e may be very large. This could yield correspondingly large values of $\bar{V}$ in Equation 1a even if the probability that primary and secondary information are not available (ε_e) is quite small.

A new parameter, the equivalent Gaussian spread (EGS), is introduced here to assist in interpreting the results of the analyses. If it is assumed that the various errors arising from the three situations represented in Equations 1a and 1b are log-normally distributed, the values of EGS was determined by the probability statement that

$$\text{Probability}\left\{ e^{-EGS} \leqslant [q_c(t)/q_T(t)] \leqslant e^{+EGS} \right\} = 0.683 \quad (2)$$

Thus, if the residuals $\ln q_c(t) - \ln q_T(t)$ were normally distributed, $(EGS)^2$ would be their variance. Because the EGS is defined so that nearly two-thirds of the errors in instantaneous streamflow data will be within plus or minus the EGS percent of the reported values, the EGS is reported here in percent.

SUMMARY OF INDIVIDUAL STUDIES

The 17 individual state studies are summarized with regard to the data use, alternative methods, and K-CERA analysis. For each step of the analysis, summary statistics are presented and an evaluation is made of what was learned. A total of 1,939 stations were analyzed in the 1983 fiscal year, which represents approximately 24 percent of the nationwide stream-gauging program.

Uses, Funding, and Availability of Continuous Streamflow Data

The analysis of data uses in the previously mentioned 17 states verified that data obtained in the national stream-gauging program are utilized for a variety of purposes by state and local governments, other federal agencies, and private industry. Of the 1,939 stations analyzed, nearly all had one or more data uses and only 35 continuous-record stations were identified as not having sufficient justification to continue their operation. In addition, 31 more short-term special study stations were identified as not having justified data uses beyond completion of their respective studies. The 66 stations that were suggested for discontinuance in the near future represent about 3 percent of the 1,939 stations analyzed.

A summary by state of the number of stations in each data-use category is given in Table 1. The data in Table 1 show that regional hydrology and hydrologic systems are the two primary data uses; 55 and 50 percent of the stations, respectively, are classified in these two categories. Streamflow data are utilized about equally in making decisions about the operation of water-resources projects, in making hydrologic forecasts of potential flooding, and in

TABLE 1 The Number of Stations in Each Data Use Category for the Stream-Gauging Program Analyzed in the 1983 Fiscal Year

State	Total No. of Stations	Regional Hydrology	Hydrologic Systems	Legal Obligations	Planning and Design	Project Operation	Hydrologic Forecasts	Water-Quality Monitor-ing	Research	Other
Alaska	110	91	40	0	36	7	21	38	17	4
Arkansas	49	34	29	1	4	34	11	23	0	1
Northern California	127	30	73	0	15	49	24	19	9	2
Central Florida	94	81	86	0	27	21	1	23	0	0
Georgia	98	64	51	0	11	34	31	36	1	0
Hawaii	124	56	65	0	1	0	0	7	0	5
Idaho	156	85	108	3	43	84	76	49	13	14
Illinois	138	88	87	0	37	17	52	96	90	0
Iowa	110	64	42	0	0	76	80	17	1	0
Kansas	140	73	88	5	2	92	114	115	0	0
Maine	51	28	16	0	0	18	26	6	8	5
Massachusetts	76	15	55	0	5	23	9	22	10	1
Nebraska	145	62	133	11	15	135	87	41	3	4
New Jersey	101	87	29	3	7	37	33	41	10	2
Pennsylvania	223	145	27	3	4	67	61	142	8	14
Rhode Island	15	5	5	0	7	1	2	10	5	0
Washington	182	66	29	4	32	37	51	57	33	62
Total	1,939	1,074	963	30	246	732	679	742	208	114

monitoring the water quality of the nation's streams. Of the stations classified, 38, 35, and 38 percent, respectively, were contained in the project operation, hydrologic forecasts, and water-quality-monitoring categories. The legal obligations and planning and design categories contained a relatively low percentage of stations (1.5 and 13 percent, respectively). The research and "other" categories were also relatively small (11 and 6 percent, respectively). The research category has decreased significantly in recent years with the completion of many small streams rainfall-runoff modeling projects and the curtailment of activity in the coal and oil-shale hydrology programs. The "other" category includes uses that do not fit into the other eight categories. Many districts included stations that were operated for recreational purposes in this category.

A funding summary for the stream-gauging program that is analyzed is given in Table 2. A shown, the primary funding source for the stream-gauging program is the COOP program. Approximately 61 percent of the stations that were analyzed in the 1983 fiscal year were financed by the COOP program. The next major category was the OFA program with approximately 32 percent of the stations. The federal and other nonfederal programs are about equal with 9 percent of the stations in each of these categories. With the exception of the other nonfederal program, the percentages reported above agree fairly well with the values given by Gilbert and Buchanan (14) for the entire 1981 USGS water-data program. They reported that 60.6 percent of the funding was provided through the COOP program, 27.3 percent through the OFA program, 11.8 percent through the federal program, and 0.3 percent through the other nonfederal program. At least from a funding standpoint, the stream-gauging program analyzed in the 1983 fiscal year appears to be fairly representative of the nationwide program.

A summary of data availability is given in Table 3. As given, data for nearly all stations are published in the annual data report of the USGS. Only 5 of 1,939 stations do not have data published in the annual report. These stations are primarily short-term stations operated for special studies. Of the stations, approximately 27 percent have data available on a real-time basis from either a satellite data-collection platform or some type of landline

TABLE 2 The Number of Stations in Each Funding Category for the Stream-Gauging Program Analyzed in the 1983 Fiscal Year

State	Total No. of Stations	Federal Program	OFA Program	COOP Program	Other Nonfederal Programs
Alaska	110	25	26	52	16
Arkansas	49	6	33	25	1
Northern California	127	3	23	67	40
Central Florida	94	0	10	84	0
Georgia	98	10	44	32	19
Hawaii	124	7	10	108	0
Idaho	156	12	58	94	0
Illinois	138	4	46	91	0
Iowa	110	18	76	52	0
Kansas	140	10	62	72	1
Maine	51	3	15	23	14
Massachusetts	76	2	16	60	0
Nebraska	145	32	26	99	0
New Jersey	101	5	12	67	22
Pennsylvania	223	20	109	169	16
Rhode Island	15	3	1	12	0
Washington	182	9	48	84	47
Total	1,939	169	615	1,191	176

Note: A single station may have multiple sources of funding. Therefore, the total number of stations may be less than the sum of the stations listed under specific programs (e.g., 110 versus 119 for Alaska).

TABLE 3 The Number of Stations in Each Data Availability Category for the Stream-Gauging Program Analyzed in the 1983 Fiscal Year

State	Total No. of Stations	Annual Report	Real Time	Provisional
Alaska	110	110	18	4
Arkansas	49	49	17	32
Northern California	127	127	17	58
Central Florida	94	94	2	6
Georgia	98	98	39	49
Hawaii	124	124	0	0
Idaho	156	156	67	44
Illinois	138	138	37	25
Iowa	110	110	76	9
Kansas	140	140	54	8
Maine	51	51	21	12
Massachusetts	76	74	18	4
Nebraska	145	145	15	75
New Jersey	101	98	32	9
Pennsylvania	223	223	39	20
Rhode Island	15	15	2	0
Washington	182	182	79	18
Total	1,939	1,934	533	373

telemetry. As can be noted in Table 3, Iowa has telemetry at almost 70 percent of their stations. It is anticipated that the percentage of stations available on a real-time basis will increase nationwide in the future.

Alternative Methods of Developing Streamflow Information

Flow-routing and regression models were developed for 145 different stations as part of the 1983 analysis of the stream-gauging program. This represents 7.5 percent of the 1,939 stations analyzed. Flow-routing methods (8) were applied to 52 stations and regression methods were applied to 129 stations for a total of 181 applications of an alternative method. There were 35 stations for which both the flow-routing and regression models were applied. Of the 145 stations analyzed, only 6 were considered to have acceptable accuracy of the simulated flows. Two of these stations were being utilized in a real-time data collection program, so they were not suggested for discontinuance. The other four stations were suggested for discontinuance conditioned on agreement from the cooperators. There were 14 additional stations where the analysts reported promising results and suggested further study to refine the models and to pursue discussions with the data users to define acceptable accuracy.

Different criteria relative to acceptable accuracy of the simulated flows were used in the individual studies. Therefore, the results reported above concerning the number of successful applications of alternative methods are not consistent across all states. A more consistent way of evaluating the alternative methods analysis would be to summarize these stations that meet certain accuracy requirements. A review of the individual state analyses indicated that 23 stations had 75 percent or more of the simulated flows within 10 percent of the observed flows for either the flow-routing model or regression model or both. Likewise, there were 13 stations that had 85 percent or more of the simulated flows within 10 percent of the observed flows and one station exceeded 95 percent. The best results for the flow-routing model were on the Rock River in Illinois (94 percent within 10 percent), the Ohio River in Pennsylvania (93 percent within 10 percent), and the Skagit River near Marblemount in Washington (93 percent within 10 percent). The best results for the regression model were also on the Skagit River (97 percent within 10 percent). These are all large rivers with relatively low variability of flow and a small percentage of intervening drainage area between stations. The application of the flow-routing or regression models on streams with a large percentage of intervening drainage area or regression modeling on nearby watersheds did not result in acceptable accuracy.

The flow-routing and regression models were both applied to 35 stations. A comparison was made of the accuracy of the two models by utilizing the results for only those stations that had at least 50 percent of the simulated flows within 10 percent of observed flows and for which the results were reported in the individual state analyses. An analysis of the 21 stations meeting these criteria revealed that the flow-routing results were most accurate for 10 stations, and the regression results were most accurate for 11 stations. It is fairly obvious that the two models are comparable in accuracy based on the assumption that each model was "adequately" calibrated for each station analyzed.

Cost-Effective Resource Allocation

Suggestions were made after the data-use and alternative-methods analyses regarding the discontinuation of stations. In some studies, the stations suggested for discontinuance were omitted from the K-CERA analysis, whereas in others they were included pending discussions with the cooperators. As a result, 1,894 stations were included in the K-CERA analysis. These stations were included on various routes and the cost of operating these stations were included in the budget. However, uncertainty functions were developed for 1,714 stations. This implies that 180 stations were not used in computing the average standard errors for the various state programs. The primary reasons that uncertainty functions could not be developed for these 180 stations were lack of discharge measurements to develop a rating curve and the inappropriateness of the data at the site to fit the basic assumptions of the current K-CERA techniques. For the current operating procedures, the average standard errors ranged from 10 to 36 percent with a weighed average standard error of 21.0 percent for all 1,714 stations analyzed. The total budget for the current operating procedure is $11,425,650. By altering the field activities as determined in the individual K-CERA analyses and maintaining this current budget, this weighted average standard error can be reduced to 19.0 percent. The current weighted average standard error of 21.0 percent can conversely be achieved with a reduced budget of $10,889,800, a total reduction of $535,850. An example of the relationship between average standard error and budget is shown in Figure 2 for Maine (4).

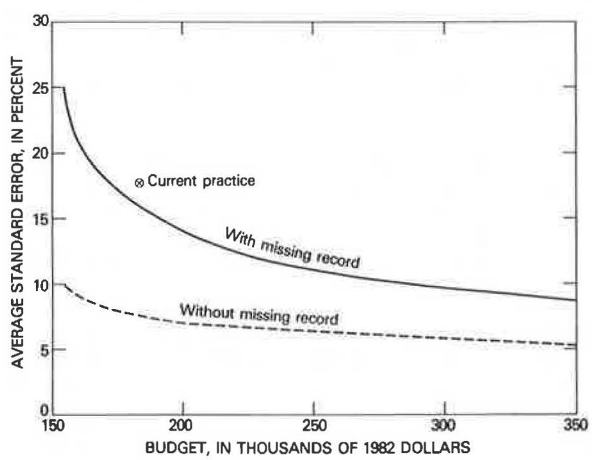


FIGURE 2 An example of the relationship between average standard error per station and budget.

Even though the EGS value was not computed for the entire stream-gauging program that was analyzed in 1983, a comparison of EGS and standard error values for an individual state analysis will illustrate the relative differences. Fontaine et al. (4) reported an average standard error of 17.7 percent and an EGS of 4.2 percent for the current operating practice. In using this study as a guideline, the weighted average standard error of 21.0 percent for the 17 studies is approximately equivalent to an EGS value of 5 percent. This comparison is predicated on the fact that the percentage of lost stage record did not vary significantly among most states. This implies that for two-thirds of the time, the error in estimating the instantaneous discharge is approximately 5 percent.

Some analysts developed an uncertainty-cost relation under the assumption that the instrumentation gave a complete stage record throughout the year. For the current budget and optimal operating practice, an analysis of nine different studies indicated a reduction in standard error of 7 percent if no missing stage record is assumed. If this reduction in standard error is indicative of all the states, this implies that the average standard error of 19.0 percent can be reduced to approximately 12 percent. It is obvious that the standard error of the missing record is a major portion of the total standard error (see Figure 2 for Maine). Nearly all analysts recognized this fact and suggested that satellite delay relay, landline telemetry, and observers be utilized to reduce the occurrence of the missing stage record.

FUTURE DEVELOPMENTS

Research and development will continue on improving the methodology for cost-effective analysis of the stream-gauging program. For several studies completed in fiscal year 1983, a sample of stations with the highest standard errors will be analyzed to determine whether the Markovian model assumed for the Kalman filter is appropriate. This analysis should provide some guidelines on when the Markovian model is appropriate.

Improved estimates of the variance of the missing record are needed because of the importance of this factor on the total standard error at a station. The present model relies primarily on correlation with nearby stations to estimate the variance of the missing record. The flows for previous days at the station of interest are not considered. New techniques for estimating the variance of the missing record should include the length of the missing period and the correlation with flows just prior to the missing period. In this way, the hydrograph-recession characteristics can be utilized to estimate the variance of the missing record. This new technique will be incorporated into the appropriate computer program in the near future.

A study is planned to investigate the percentages of the missing record used in the K-CERA analysis to determine whether they are realistic. Because of the time limitations of the individual studies, most analysts were unable to do a detailed analysis of the missing record issue. A more detailed analysis is important because of the sensitivity of the total standard error to this factor.

Research is also continuing on more sophisticated models for the Kalman filter. As of 1984, work was underway to develop models for describing Kalman filters that are appropriate for sand channel streams and streams where artificial controls are regularly cleaned. These models will be used in studies as soon as they are operational. Every attempt is being made to utilize all information normally available to the analyst who computes the published record.

New studies initiated in fiscal year 1984 are utilizing the improvements noted in this paper as they become available. The primary objective of the present research is to make the K-CERA package of programs an effective tool for managing and determining the accuracy of data that is generated through the nationwide stream-gauging program.

REFERENCES

1. N.E. Hutchison. WATSTORE: National Water Data Storage and Retrieval System of the U.S. Geological Survey. User's Guide, Vol. 1. Open-File Rept. 75-0426. U.S. Geological Survey, Denver, Colo., 1975, 791 pp.
2. M.A. Benson and R.W. Carter. A National Study of the Streamflow Data-Collection Program. Water-Supply Paper 2028. U.S. Geological Survey, Washington, D.C., 1973, 44 pp.
3. M.E. Moss and E.J. Gilroy. Cost Effective Stream-Gauging Strategies for the Lower Colorado River Basin. Open-File Rept. 80-1048. U.S. Geological Survey, Reston, Va., 1980, 128 pp.
4. R.A. Fontaine, M.E. Moss, J.A. Smath, and W.O. Thomas, Jr. Cost-Effectiveness of the Stream-Gauging Program in Maine. Water-Supply Paper 2244. U.S. Geological Survey, Washington, D.C., 1984, 39 pp.
5. V.B. Sauer. Unit Response Method of Open-Channel Flow Routing. Journal of the Hydraulics Division, ASCE, Vol. 99, No. HY1, 1973, pp. 179-193.
6. T.N. Keefer. Desktop Computer Flow Routing. Journal of the Hydraulics Division, ASCE, Vol. 100, No. HY7, 1974, pp. 1047-1058.
7. T.N. Keefer and R.S. McQuivey. Multiple Linearization Flow Routing Model. Proc., ASCE, Vol. 100, No. HY7, 1974, pp. 1031-1046.
8. W.H. Doyle, H.O. Shearman, G.J. Stiltner, and W.O. Krug. A Digital Model for Streamflow Routing by Convolution Methods. Water-Resources Investigations Rept. 83-4160. U.S. Geological Survey, Denver, Colo., 1983, 136 pp.
9. N.R. Draper and H. Smith. Applied Regression Analysis, 2nd ed. John Wiley and Sons, New York, N.Y., 1966.
10. D.G. Kleinbaum and L.C. Kupper. Applied Regression Analysis and Other Multivariable Methods. Duxbury Press, North Scituate, Mass., 1978.
11. H.C. Riggs. Regional Analysis of Streamflow Characteristics. Techniques of Water-Resources Investigations Book 4. U.S. Geological Survey, Alexandria, Va., 1973, 15 pp.
12. A. Gelb. ed. Applied Optimal Estimation. Massachusetts Institute of Technology Press, Cambridge, Mass., 1974.
13. E.J. Gilroy and M.E. Moss. Cost-Effective Stream-Gauging Strategies for the Lower Colorado River Basin. Open-File Rept. 81-1019. U.S. Geological Survey, 1981, 128 pp.
14. B.K. Gilbert and T.J. Buchanan. Water-Data Program of the U.S. Geological Survey. Circular 0863. U.S. Geological Survey, 1982, 16 pp.

Hydrologic Research on Coastal Plain Watersheds of the Southeastern United States

J. M. SHERIDAN and W. C. MILLS

ABSTRACT

The Southeast Watershed Research Laboratory (SEWRL), of the Agricultural Research Service, U.S. Department of Agriculture, is conducting hydrologic research studies on watersheds in the Coastal Plain of the southeastern United States. The Coastal Plain is a region where extensive hydrologic data bases have generally not been available because of difficulties associated with the accurate measurement of streamflow in the low-gradient channel systems of the broad, heavily vegetated floodplains. The SEWRL has a 129-square mile drainage area, the Little River Watershed (LRW), which is divided into seven subwatersheds that are instrumented to obtain hydrologic data (rainfall, streamflow, and alluvial groundwater) for use in analyzing and evaluating Coastal Plain hydrologic processes. A description of the experimental study areas and the associated hydrologic instrumentation is presented. Basic hydrologic information is presented, as well as flood design information including instantaneous peak flow and maximum mean daily flow relationships developed from the LRW hydrologic data. Ratios of instantaneous peak flows to maximum mean daily flows for selected return intervals for watersheds of 1 to over 100 square miles are also presented. Additionally, an evaluation of the application of the Cypress Creek procedure (commonly used for agricultural drainage design) on two LRW subwatersheds is presented.

The Southeast Watershed Research Laboratory (SEWRL) of the Agricultural Research Service, U.S. Department of Agriculture (USDA-ARS) in Tifton, Georgia, is conducting hydrologic research studies on watersheds in the Coastal Plain of the southeastern United States. The SEWRL has instrumented as its primary study area a 129-square mile watershed, the Little River Watershed (LRW). This watershed is considered to be generally representative of the Southern Coastal Plain Land Resource Area (1). The Southern Coastal Plain is a rather extensive, agriculturally important region. It is also a region where accurate hydrologic data bases have generally not been available because of the difficulties associated with the measurement of streamflow in the low-gradient channel systems of the broad, heavily vegetated floodplains that are characteristic of the region.

The LRW was instrumented to provide data for analyzing and evaluating Coastal Plain hydrologic processes and for the development and testing of conceptually based prediction methodologies for use on ungauged watersheds in low-relief physiographic regions. In addition to the original hydrologic objectives, these facilities are also providing a valuable data base for the SEWRL and other ARS scientists and their cooperators for erosion and water quality modeling.

This paper presents an overview of the LRW experimental study areas, and the associated hydrologic instrumentation, as well as some basic hydrologic data and flood flow analyses from these watersheds.

STUDY AREA DESCRIPTION

Location and Topography

Little River originates 6 miles west of Ashburn, Georgia, and flows in a southerly direction, to its confluence with the Withlacoochie River, then to the Suwanee River, eventually emptying into the Gulf of Mexico. The instrumented portion of the LRW includes Little River and its tributaries from its headwaters downstream to approximately 4 miles west of Tifton, Georgia--a drainage area of about 129 square miles. The LRW is located in Tift, Turner, and Worth Counties, Georgia, and is divided into seven subwatersheds ranging from approximately 1 to 45 square miles. Figure 1 shows the location of the experimental study area within the Southern Coastal Plain Land Resource Area.

Topographically, LRW is an area of floodplains, river terraces, and gently sloping uplands. Woodruff (2) described the area as "one of low relief; a gently undulating surface of broad interfluves and shallow valleys." Valley bottoms are nearly level, and valley side slopes are generally less than 5 percent, although some range from 5 to 15 percent. Floodplains range in width from 200 ft to 0.5 mile (3). Surface elevations within the watershed range from about 260 to 470 ft above mean sea level.

Geology, Soils, and Vegetation

The Coastal Plain province of the United States extends from New England in the Northeast, south along the Atlantic Coast, and then west into Texas (4). LRW lies within the Tifton Upland subprovince of the southeastern Coastal Plain. The Tifton Upland is defined by the outcrop area of the Miocene series, Hawthorne Formation, which is the surface formation in most of the Tifton Upland. The Hawthorne Formation is overlain by loose, unconsolidated Quaternary and Recent sediments that form shallow phreatic aquifers. The Hawthorne Formation is an aquiclude of low vertical transmissibility that should yield very little surface water to deep aquifers (5). A shallow water table exists throughout the watershed at

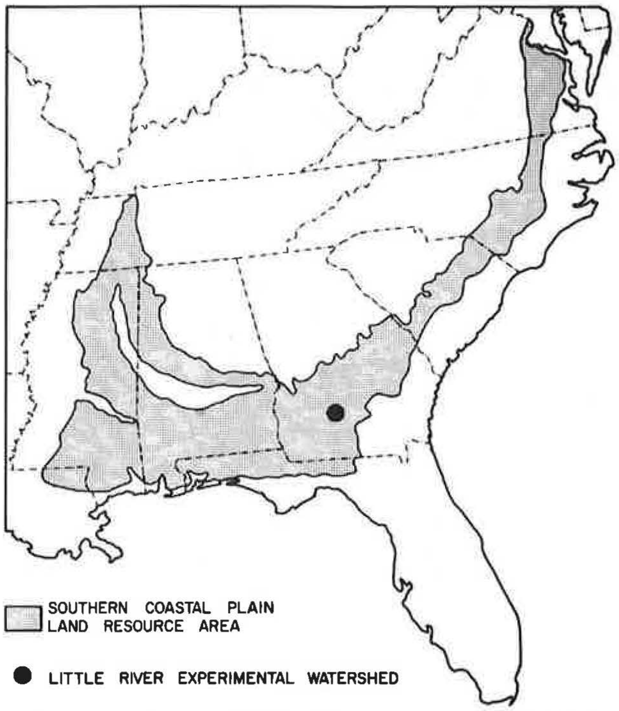


FIGURE 1 Location of LRW within the Southern Coastal Plain Land Resource Area.

depths of 0 to 19 ft. Depths to the water table along drainage divides range from 9 to 19 ft, and generally decrease toward the major stream (5).

Soils of this region have been formed from materials of the Miocene and possibly the Pliestocene age. The upland soils developed in place whereas most materials on stream terraces and bottoms of creeks and rivers were derived from alluvium washed from the upland Coastal Plain soils (3).

Soils of the watershed are predominantly sandy and light-colored with high infiltration rates (6). At depths of 3 to 20 ft, a relatively impermeable material described as plinthite greatly restricts downward movement of soil water, resulting in perched water tables. Internal drainage of most upland soils is good, but that of the swamp-alluvial soils is poor to very poor, with water standing on the surface during portions of the year (7).

The native upland vegetation of the LRW (longleaf, pine/perennial wiregrass) has been almost totally replaced by row crops, pastures, pine plantations, roads, and residential and commercial properties (8). A transitional area of hardwood pine generally occurs between the dry uplands and the wet bottomlands. A dense undercover is characteristic of this community, which is generally found on the Alapaha soil series (9). The bottomland or riparian areas of the LRW are classified as Blackwater swamp systems (10). Swamp hardwood communities occur along stream edges--the canopy is closed, and the undergrowth is thick. This community is characteristic of the alluvial soil series (9).

General Hydrology

Precipitation in the Tifton Upland occurs almost exclusively as rainfall. During the winter and early spring months, events are characterized by widespread frontal storm activity. During the late spring and summer months, convective thunderstorm activity often produces short duration rainfall events with high intensities--frequently of a localized nature. Intensities during these events may exceed the generally high infiltration capacities of Coastal Plain soils for short time intervals.

Precipitation data from the Coastal Plain Experiment Station (CPES) for 1923-1983 show a mean annual rainfall of 47.41 in. with a standard deviation of 8.73 in. Observed extremes of annual rainfall were 23.25 in. (1954) and 70.90 in. (1928). Average monthly precipitation amounts are well distributed throughout the year except for the fall, with average monthly totals exceeding 3.5 in. except for October. Although monthly averages are well distributed, actual monthly rainfall totals show wide variation. Significant rainfall-deficient periods may occur during all seasons of the year (11).

The occurrence of the Hawthorne Formation (an aquiclude) and the high infiltration characteristics of the upland soils of the region result in conditions that are conducive to subsurface movement of significant quantities of infiltrated precipitation from upland areas and valley flanks to the stream systems. Instrumented upland areas have shown that approximately 80 percent of the flow moving from those areas is subsurface (12,13). This is confirmed by estimates of base flow or delayed subsurface flow from LRW watersheds that ranged from 58 to 82 percent (14,15). This prolonged subsurface flow from the uplands results in high water tables or standing water in the low-lying, poorly drained areas of these watersheds for up to 3 weeks after streamflow ceases (16). This further results in a saturated or high water table zone that is believed to conform to the source area theory of runoff that has evolved in the work of numerous researchers.

During prolonged periods of low or deficient rainfall, streamflow typically ceases on the smaller streams and rivers, generally during the late summer and fall months. During this time, water in the poorly drained areas is depleted by evapotranspiration and these areas, which usually function as high runoff-producing zones during storm events, may become zones of high water storage capacity for incident rainfall, as well as for surface and subsurface runoff from valley flanks and adjacent uplands.

HYDROLOGIC INSTRUMENTATION

The original hydrologic monitoring network on LRW included precipitation measurement at approximately 55 sites, 8 streamflow measuring sites, and alluvial groundwater measurements at 3 channel cross sections. Some of these measurement sites, however, have been discontinued in recent years. Figure 2 shows the LRW study areas and the location of hydrologic instrumentation for the measurement of rainfall, streamflow, and alluvial groundwater on these watersheds. Construction and/or installation of instrumentation on LRW began in 1967 and was completed in 1972.

Rainfall Measurement

Rainfall on the LRW is measured by digital rain gauges that record cumulative rainfall totals to the nearest 0.1 in. Each gauge consists of a collector for catching and storing rainfall, a device for weighing the water, and a recording mechanism that punches a binary decimal code on paper tape at 5-min intervals. The rain-gauge network provides denser spacing of gauges on the smaller subwatersheds at the upper end of the main basin, and more sparse spacing at the lower end of the basin. This spacing was designed to provide more accurate measurement of rainfall on the smaller drainage areas where localized storms could cause extreme variability in storm runoff.

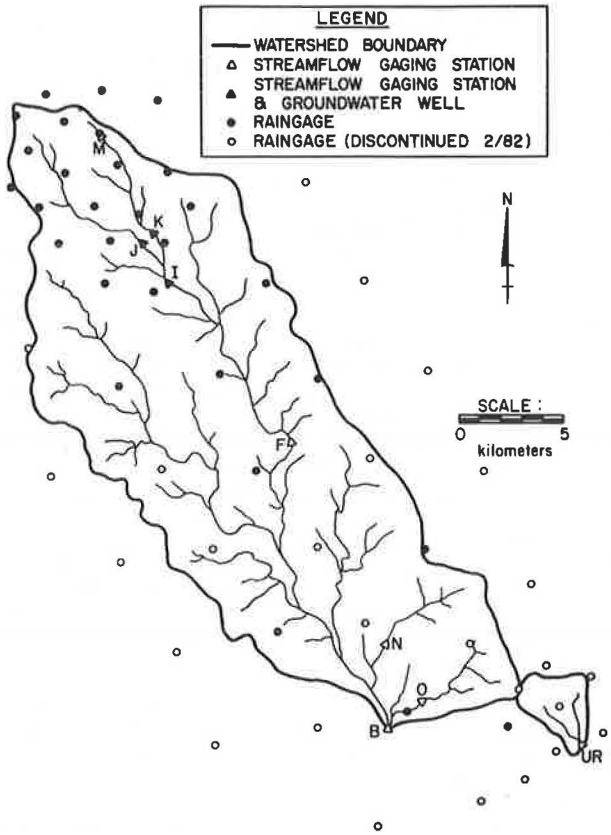


FIGURE 2 Location of hydrologic instrumentation on LRW.

Streamflow Measurement

The low-gradient drainage systems of the Coastal Plain region of the southeastern United States present a particular challenge in the measurement of stream discharge. This region is typified by broad floodplains with very poorly defined stream channels. Channel-bed slopes are generally less than 0.1 percent, and channels are distinguishable only at extremely low flow rates. The floodplains are heavily vegetated, and at moderate-to-high flow rates, discharge is spread over the entire floodplain--a width of several hundred feet. Therefore, the only practical location to confine flows for measurement is at highway bridges or culverts, which are built into raised roadbeds that transect the floodplain. Any control device that is used must be capable of withstanding some degree of submergence during substantial periods of operation while maintaining acceptable levels of gauging accuracy.

A flow measurement device referred to as a Virginia V-notch weir (17) was selected. The measurement control is a horizontal weir with a V-notch center section. This weir, although not a true broad-crested weir, does not exhibit the sensitivity to submergence of sharp-crested weirs. It also provides accuracy of measurement at low flows that is typical of the V-notch configuration, and is relatively maintenance free in operation.

Flow measurement sites selected are shown in Figure 2. These sites include five structures (M, K, I, F, and B) located in series on one of the main stems of the experimental study area. Watersheds defined by these sites in series range in area from 1.0 to 129.05 square miles. In addition, Stations J and K, and O and N provide parallel pairs of subwatershed study areas that range from 6.05 to 8.54 square miles.

Flow measurement control structures were designed to contain all flow within the V-notch portion of the weir approximately 90 to 95 percent of the time, which represents approximately 65 to 70 percent of the total flow volume (18). The structures were also designed to accurately measure the estimated 25-yr peak flow rate without exceeding the physical limitations of the control.

Each structure consists of a sheet steel piling cutoff and support wall capped by a reinforced concrete weir cap, a combination stilling-well and recorder shelter, an energy-dispersing apron, and a footbridge for use in making high-flow measurements.

Digital stage-recording devices provide continuous data on water stage elevations, both upstream and downstream. These digital recorders punch water stage evaluation in 0.01-ft increments at 5-min intervals. In addition to the two digital recorders originally installed at each site, an analog recorder was later installed to provide a backup record for the digital data.

For additional details on structural design and actual construction methods, the reader is referred to discussions by Yates (18-20), Yates and Sheridan (21,pp.345-352), and Mills et al. (22).

Alluvial Groundwater Measurement

Monitoring of alluvial groundwater was accomplished through the use of observation wells drilled into the floodplain alluvial material at three locations near streamflow measurement sites I, J, and K. Weekly manual observations were initiated on these wells in 1967, and digital stage recorders were subsequently installed in 1969 to provide continuous recording of alluvial groundwater stage.

Rates of movement of groundwater through the valley alluviums were determined with Darcy's equation for flow through porous media, using the estimated saturated alluvial cross-sectional area, estimated valley contact slope, and aquifer hydraulic conductivities obtained by conducting alluvial-well pumping tests. Results of these analyses indicate that despite the highly permeable alluvial floodplain material, and the typically large cross-sectional alluvial areas, virtually all runoff moving from these watersheds was monitored by measurement of surface runoff in the channel systems and that less than 0.01 percent of the total water budget was estimated to be allocated to alluvial subsurface water movement (23).

HYDROLOGIC DATA SUMMARY

Annual Rainfall

For the LRW period of record (1968-1983), the mean annual rainfall measured at the CPES at Tifton was 47.88 in. (standard deviation was 4.93 in.), with extreme recorded amounts of 38.38 in. and 55.30 in. For this period, the recorded average annual rainfall at the CPES was slightly greater than the long-term CPES mean. Annual amounts for 1968-1983 exhibited less variability than the long-term record, as indicated by the standard deviation, which is approximately one-half of the standard deviation of the long-term record.

Watershed weighted-average annual rainfall totals for the entire LRW, computed using the reciprocal distance squared technique (24) for the 1968-1983 period averaged 49.48 in., or more than 1.5 in. greater than the mean annual rainfall measured at the CPES. Individual subwatershed weighted-average annual totals were generally 2 to 4 in. greater than the CPES annual totals. Only 3 of 16 yr showed subwatershed totals less than the CPES annual rainfall total, whereas for 12 of the 16 yr, weighted subwatershed totals exceeded the CPES annual totals. For 1979, the upper LRW subareas averaged 8 to 10 in. more rainfall than the CPES total.

Annual Water Yield

Preliminary analyses of streamflow data from Watershed M (the 1-square mile drainage area) indicated that all runoff from that watershed may not be passing through the flow measurement device, and that some flow may instead by bypassing through an undetermined route. Cursory examination of the flow data indicated that although stormflow volumes from Watershed M were similar to those from other watersheds, the base flow volumes were significantly lower. For this reason, Watershed M water yield data were not included in these analyses. Peak flow or storm event data for Watershed M were, however, included in the flood event analyses because surface runoff boundaries are determinable.

Annual water yields for 1968-1983 on LRW subareas ranged from 1 in.[2] to nearly 30 in.[2] of runoff. This range of variability is evident in a period in which annual rainfall variability is less than typical. Annual yields by watersheds are shown in Table 1. Because available record periods are not the same for all watersheds, a common record period (1972-1981) was selected to minimize the effects of year-to-year variation in rainfall totals. Both the water yield and the percent water yield are given. Percent water yields were computed because differences in rainfall totals between upper and lower watersheds were observed. For comparison, the precipitation and water yield data from Watershed Z, an 0.85-acre, instrumented, upland cropped area located on the CPES, was included in Table 1. The water yield for Watershed Z includes both surface and subsurface measured flows.

TABLE 1 Average Annual Water Yield, Percent Water Yield, and Mean Annual Flow Rates by Watersheds for Common Record Period (1972-1981)

Watershed	Area (square miles)	Average Annual Water Yield (in.)	Water Yield (%)	Mean Annual Flow Rate[a] (ft^3/sec)	Mean Annual Flow (ft^3/sec/mile)
I	19.38	15.95[a]	31.8[a]	22.75	1.17
J	8.54	15.50[a]	30.7[a]	9.74	1.14
K	6.43	15.19[a]	30.3[a]	7.19	1.12
N	6.05	14.28[a]	29.0[a]	6.36	1.05
F	44.34	14.00[a]	28.3[a]	45.69	1.03
O	6.15	13.67[a]	28.0[a]	6.19	1.01
Z	0.0013	13.68[a]	28.8[a]	0.0013	1.03
B	129.05	12.93[a]	26.1[a]	122.82	0.95

[a]Means with the same letter are not significantly different at the 5 percent level for Duncan's Multiple Range Test.

The mean annual water yields were also converted to mean annual flow rates for the respective watersheds. Water yields and mean annual flow rates per square mile appear to be relatively independent of area, although the flows from Watershed B are somewhat lower than those of all subwatersheds.

FLOOD DESIGN INFORMATION

Instantaneous Peak Flows

Design of highway and agricultural drainage structures requires reliable estimates of peak flow rates from ungauged areas for selected return periods. To evaluate peak flow data and develop information suited to making design estimates for Coastal Plain watersheds, a frequency analysis was performed on annual (water year) peak flow rates for each watershed using the total available record. The log-Pearson Type III probability distribution was fitted to the LRW annual peak flows using procedures that were recommended by the U.S. Water Resources Council (25), and that were contained in computer programs developed by the Soil Conservation Service (SCS), U.S. Department of Agriculture (personal communication with Roger Cronshey).

The U.S. Geological Survey (USGS), in cooperation with the Georgia Department of Transportation, in a study on flood flows on small (<20 square miles), rural streams in Georgia, has published regression equations for estimating peak flows developed from 10 yr of extensive flow measurements on small watersheds in Georgia (26). Multiple regression analyses were performed by Golden and Price on a regional basis to evaluate 10 climatological and basin parameters.

Their analyses indicated that drainage basin size was the most significant predictor and that other parameters did not significantly decrease the standard error. For the Coastal Plain region of Georgia, two subregions were identified by this regressional analysis as having a geographic bias. These two regions were the Sand Hills of Georgia (a narrow belt across the state that separates the Coastal Plain from the Piedmont, which is characterized by soils with extremely high infiltration rates) and the Ochlocknee basin, an area of high flood runoff located in southeast Georgia near the Georgia-Florida boundary. Elimination of these areas left in 105 stations that were used by USGS in the development of regression equations for peak flows in the Coastal Plain region.

The USGS regression equations are shown in Table 2 along with regression equations developed from frequency analyses on the LRW data. For all return intervals, the regressions developed from the LRW data have somewhat lower coefficients than the USGS regressions. For the shorter return intervals, the LRW regression exponents are higher than the USGS values, whereas the exponent is the same at the 10-yr recurrence interval. For the longer return periods (25 and 50 yr), exponents for the LRW regressions are slightly lower than the USGS regression exponents.

TABLE 2 Summary of Regression Equations for Instantaneous Peak Flows

Recurrence Interval (yr)	LRW	USGS Region 3 *(25, 26)*
2	$86\ A^{0.69}, r^2 = 0.98$	$99\ DA^{0.58}$
5	$145\ A^{0.63}, r^2 = 0.97$	$167\ DA^{0.59}$
10	$190\ A^{0.59}, r^2 = 0.96$	$216\ DA^{0.59}$
25	$252\ A^{0.56}, r^2 = 0.95$	$280\ DA^{0.59}$
50	$302\ A^{0.53}, r^2 = 0.94$	$332\ DA^{0.60}$

Note: DA = the drainage area of the watershed; LRW includes 8 coastal plain watersheds and USGS Region 3 includes 105 coastal plain watersheds.

The estimated peak flow rate for the 25-yr event (the design return interval for the LRW structures) is plotted for each subwatershed in Figure 3. The USGS regression for the 25-yr return period for Region 3 (the Coastal Plain) is also plotted for comparison. There is generally good correspondence between the fitted 25-yr peak flow rates for the LRW and the USGS regressions, although the LRW values are somewhat lower than those estimated with the

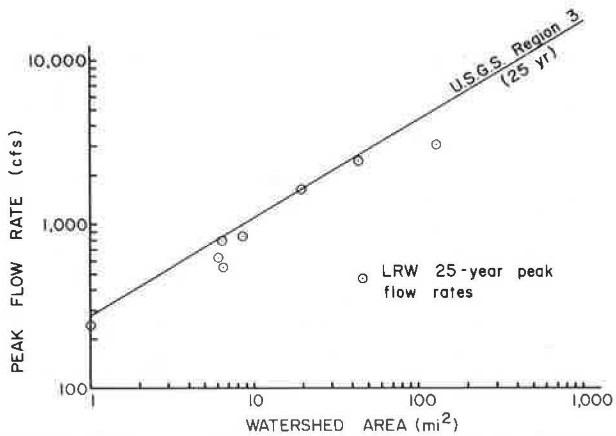


FIGURE 3 LRW 25-yr instantaneous peak flows (as determined by frequency analyses) and peak flows predicted by USGS regression for 25-yr return period.

USGS regression. The 25-yr peak flow rate for Watershed B, the total Little River drainage basin, is particularly low. An evaluation of spatial rainfall distribution for major events on Little River has shown that for most of these events, the largest amount of rain occurred on the upper portion of the watershed (22). This lack of total coverage of the basin by major events is a possible cause of reduced peak flows at the lower end of the watershed.

Another factor that could account for some deviation between the LRW and the USGS regressions for Region 3 is that Little River represents a single Coastal Plain drainage basin. The USGS regressions were developed from data collected from a number of Coastal Plain basins.

Also, the original work by Golden and Price (26) cautioned that use of developed regressions should be limited to watersheds of less than 20 square miles in area. However, a more recent report by Price (27) contained regressions that were applicable for watersheds with drainage areas of 0.1 to 1,000 square miles, the regression parameters that were recommended for Region 3 were unchanged from those reported earlier (26).

Maximum Mean Daily Flows

Regression analyses were performed to relate drainage area to observed maximum mean daily flow rate for subwatersheds on the LRW. Maximum mean daily flows were determined for selected return intervals by fitting of the log-Pearson Type III distribution to annual (water year) maximum mean daily peaks. Although the maximum mean daily flow rate is not identical to the maximum 24-hr flow rate used by the SCS in design of drainage systems for agricultural areas, it approaches the 24-hr maximum for larger watersheds and should provide useful information for agricultural drainage design. Regressions developed for predicting maximum mean daily flows by return frequency are shown in Table 3.

Fitted regression exponents for the Coastal Plain watersheds are very close to 5/6, or 0.833, the exponent of the Cypress Creek formula, which is used by the SCS and others in design of agricultural drainage systems. The Cypress Creek formula was apparently originated by McCrory et al. (28) in Arkansas. Subsequent work by Stephens and Mills (29) in the Florida Flatwoods has confirmed that this is also a reasonable exponent for use in runoff design estimates for flatwoods areas. Regression coefficients shown in Table 3 are within the range of C values reported for the Florida watersheds of 20 to 130 for storm events with estimated return frequencies of from 2 to 50 yr.

TABLE 3 Summary of Regression Equations for Mean Maximum Daily Flows

Recurrence Interval (yr)	Regression for LRW Fitted Max. (MDQ)	Coefficient of Determination (r^2)
2	30 $DA^{0.89}$	0.99
5	45 $DA^{0.86}$	0.98
10	54 $DA^{0.84}$	0.98
25	66 $DA^{0.83}$	0.97
50	75 $DA^{0.82}$	0.97

Note: DA = the drainage area of the watershed; MDQ = mean daily discharge.

Stephens and Mills (29) also developed a relationship for estimating C values for Florida Flatwoods watersheds based on excess precipitation. Computed C values for storm runoff data from Watersheds K and O are plotted for comparison with the Florida regression for determining C values in Figure 4. LRW C values were computed by dividing the maximum mean daily discharge (MMDQ) by the drainage area (DA) raised to the 5/6 power. Thus, $C = MMDQ/DA^{5/6}$. The C value was then plotted versus the excess rainfall (i.e., the measured storm runoff).

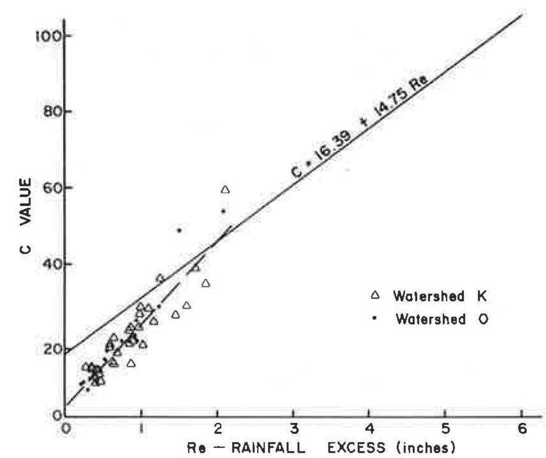


FIGURE 4 Comparison of C-values computed for LRW with C-value relationship developed for Florida Flatwoods.

As can be seen in Figure 4, the LRW C values for low excess rainfall volumes appear to be lower than those estimated by the Taylor Creek regression. However, at about 2 in. of excess rainfall, the relationships are approximately the same. Because storm event excess rainfall was about 2 in. or less for the LRW data, a regression is not presented for design use because excess amounts greater than 2 in. would require considerable extrapolation beyond the range of observed values.

Application of Cypress Creek Procedure on the LRW

Simple regression relationships are a good means of estimating design flow rates based on the single variable drainage area. However, in practice, some watersheds show differences in peak flood flows that are not explained by differences in drainage area alone. An example of this is the case of Watersheds K and O on Little River, the smaller of which, Watershed O (6.15 square miles), shows substantially larger peaks than the larger Watershed K (6.43 square miles).

The agricultural drainage design procedure that incorporates the Cypress Creek formula allows for the estimation of differences in runoff volume, and, consequently, peak 24-hr flows that are caused by differences in soil characteristics and land use. To evaluate the use of this procedure in making design discharge estimates for Coastal Plain watersheds, a sample computation was made on Watersheds K and O.

The design procedure for agricultural drainage systems (30) starts with the selection of the return frequency of the storm event and the estimation of rainfall amount (31). The SCS runoff curve number procedure (32) is then used to estimate the volume of storm runoff, or excess rainfall. The excess rainfall (Re) is then used to determine the C value based on the relationship

$$C = 16.39 + 14.75\ Re$$

that was developed by Stephens and Mills (29). This C value can then be used in the Cypress Creek formula, $Q = CM^{5/6}$ where M is the drainage area in square miles, to compute the maximum 24-hr runoff. As previously indicated, relationships developed on the Little River data indicate that the 5/6 exponent is a reasonable value for use on the Coastal Plain watersheds.

For Watersheds K and O, design 24-hr rainfall amounts (31) of 4.0, 5.3, 6.3, 7.3, and 8.0 in. were determined for 2-, 5-, 10-, 25-, and 50-yr return intervals, respectively. These totals were then multiplied by a 0.97 factor for depth-area reduction. Excess rainfall, or storm runoff amounts, was then computed using the SCS runoff curve number procedure (32).

Effective runoff curve numbers for these two watersheds were computed based on the percent area of each of the watersheds in selected soil types (i.e., B and D hydrologic soil groups), and the percent area in selected land use categories (lowland forest, agricultural cropland, and upland forest). One departure from conventional procedure was made. For the alluvial floodplains only, the average antecedent condition was assumed to be a wet, or AMC III, condition. It is believed that this assumption is justified because, as discussed in the general hydrology section of this paper, the delayed subsurface flow from upland areas in these watersheds results in saturated or high water table areas within the floodplain for major portions of the year. Use of this assumption on these two watersheds, where floodplain/alluvial soils account for 22 and 28 percent of the total watershed areas, resulted in increases in the effective average runoff-producing condition curve number of 2.5 and 4.0. Runoff curve numbers were computed for high, average, and low runoff-producing antecedent conditions, and excess rainfall (storm runoff) was determined graphically (32).

Results of this computation for the two watersheds (shown in Table 4) indicate that the use of the average runoff-producing condition for determining excess rainfall and then determining C value based on this excess rainfall estimate using the regression developed for the Florida Flatwoods by Stephens and Mills (29) may lead to significantly underpredicted maximum mean daily design discharge rates for Coastal Plain watersheds. This is true particularly for the shorter return interval storms, which are the primary application of the Cypress Creek formula in agricultural drainage design. This underestimation occurred even with the use of the wet condition as the average antecedent condition for the floodplains. Use of the high runoff-producing antecedent condition overpredicted the maximum mean daily flow rate.

This observed tendency is believed to be caused by the use of design return period rainfall to predict runoff for a comparable return period. Rainfall for any specified return interval may fall on a watershed of high runoff-producing or wet antecedent condition, or it may fall on a watershed of average or of low run-off producing conditions. As can be seen in Table 4, for these Coastal Plain watersheds, the estimated runoff volume from a 50-yr rainfall

TABLE 4 Comparison of Estimated MMDQ for High, Average, and Low Runoff-Producing Conditions with MMDQ Obtained from Frequency Analyses on Two Little River Subwatersheds

Watershed K						Watershed O					
	Return Period (years)						Return Period (years)				
	2	5	10	25	50		2	5	10	25	50
24-hr RF (in.)	4.00	5.30	6.30	7.30	8.00	24-hr RF (in.)	4.00	5.30	6.30	7.30	8.00
x 0.97 (in.)	3.88	5.14	6.11	7.00	7.76	x 0.97 (in.)	3.88	5.14	6.11	7.00	7.76
High-Runoff-Producing Conditions (CN-71)						High-Runoff-Producing Conditions (CN-78)					
RO (in.)	1.32	2.22	3.00	3.75	4.35	RO (in.)	1.80	2.81	3.70	4.58	5.20
C-value	36.0	49.0	60.5	71.5	80.5	C-value	43.0	58.0	71.0	83.5	93.0
MMDQ	169.6	230.9	285.1	336.9	379.3	MMDQ	195.2	263.4	322.4	379.2	422.3
Average-Runoff-Producing Conditions (CN-53)						Average-Runoff-Producing Conditions (CN-59)					
RO (in.)	0.40	0.95	1.45	1.98	2.40	RO (in.)	0.65	1.30	1.90	2.52	3.02
C-value	22.5	30.5	38.0	45.5	51.5	C-value	26.0	35.5	44.0	53.5	61.0
MMDQ	106.0	143.7	179.1	214.4	242.7	MMDQ	118.1	161.2	199.8	242.9	277.0
Low-Runoff-Producing Conditions (CN-47)						Low-Runoff-Producing Conditions (CN-51)					
RO (in.)	0.25	0.58	1.00	1.45	1.88	RO (in.)	0.35	0.80	1.28	1.78	2.20
C-value	20.0	25.0	31.0	38.0	44.0	C-value	21.5	28.0	35.0	42.0	48.5
MMDQ	94.2	117.8	146.1	179.1	207.3	MMDQ	97.6	127.1	158.9	190.7	220.2
MMDQ (frequency analysis)	159	207	234	264	284	MMDQ (frequency analysis)	145	197	232	277	310

Note: RF = rainfall; RO = runoff.

event with low runoff-producing antecedent conditions will be exceeded by the runoff volume from a 5-yr storm event on a watershed with high runoff-producing conditions.

Ratio of Instantaneous Peak Discharge to Maximum Mean Daily Flow

Ratios of instantaneous peak discharge to maximum mean daily discharge were computed for the respective return periods for each of the subwatersheds. A regression of this ratio on the log transform of drainage area gave good results, with r^2 ranging from 0.91 to 0.99 for the 2- to 50-yr return frequencies. These fitted regressions permit the generation of a family of curves (see Figure 5) that may be used for converting estimated maximum mean daily flow rate to peak instantaneous discharge for any size drainage area from approximately 1 to over 100 square miles, and for any of the specified return intervals.

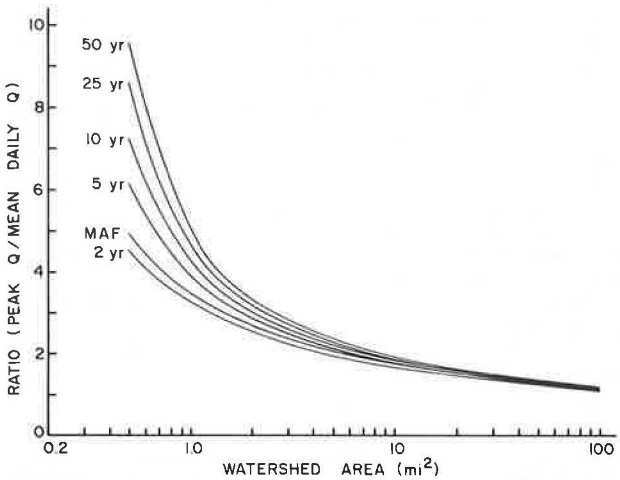


FIGURE 5 Curves for estimating ratios of instantaneous peak flow to maximum mean daily discharge based on watershed area for selected return periods.

The derived ratios for the Coastal Plain (LRW) data were compared with ratios (Q_i/Q_{24}) developed by Stephens and Mills (29) for the Florida Flatwoods watersheds. The average ratios for the Flatwoods watersheds ranged from 1.54 for a 10-square mile watershed down to 1.14 for a 100-square mile watershed. Ratios for the Caostal Plan watersheds ranged from about 1.80 for a 10-square mile watershed down to about 1.15 for a 100-square-mile watershed for the mean annual flood, MAF (2.33-yr return period).

The family of curves developed on the LRW data gives considerable additional capability by providing ratios for making estimates of instantaneous peak flows from Coastal Plain watersheds based on maximum mean daily flow rates for a range of return periods. These curves also extend to drainage areas of under 10 square miles, which provide conversion values down to 1 square mile.

CONCLUSIONS

The following conclusions may be observed:

1. Observed annual water yields on LRW Coastal Plain watersheds ranged from 1 to nearly 30 in.^{2}, which averaged about 13 in^2 for the total watershed (129.05 square miles) for the record period 1972-1983. Relative yields averaged from 26 to 32 percent of annual rainfall.
2. Peak instantaneous flow rates from the LRW subwatersheds generally fit the available regional USGS regressions, although the peak flow from the total 129.05-square mile drainage area was particularly lower than the predicted value using the USGS regression.
3. Maximum mean daily flows regressed on drainage area generally conformed to the 5/6 exponent in the Cypress Creek formula that has been widely used for agricultural drainage system design.
4. Computed C values for the LRW appear to be lower for low excess rainfall amounts than those estimated by using the available regression developed on Florida Flatwoods watersheds. Sufficient data are not available to develop a relationship between excess rainfall and C value for the LRW data.
5. Use of an average runoff-producing antecedent watershed condition underpredicted observed maximum mean daily flow rates for two subwatersheds of LRW, particularly for the shorter (2-5 yr) return intervals predicted by using the Cypress Creek formula and the regression of the excess rainfall and C value developed for Flatwoods watersheds in Florida. Use of the high runoff-producing condition overestimated observed maximum mean daily flow rates.

REFERENCES

1. Land Resource Regions and Major Land Resource Areas of the United States. Agriculture Handbook 296. Soil Conservation Service, U.S. Department of Agriculture, 1981, 156 pp.
2. J.F. Woodruff. Some Geomorphic Aspects of the Upper Little River Watershed, Tift, Turner, and Worth Counties, Georgia. In Progress Report of Hydrology Study of Upper Little River Watershed. Univ. of Georgia; U.S. Department of Agriculture, Tifton, 1968.
3. E.R. Jensen, L.E. Aull, J.L. Shepard, C.B. Thomas, R.L. Carter, E.S. Haygood, and R.G. Middleton. Soil Survey of Tift County, Georgia, No. 3. Soil Conservation Service, U.S. Department of Agriculture, 1959, 28 pp.
4. N.M. Fenneman. Physiography of the Eastern United States. McGraw-Hill Book Company, Inc., New York, 1970, 689 pp.
5. R.E. Carver. Reconnaissance Geology of the Little River Watershed, Tift, Turner, and Worth Counties, Georgia. In Progress Report of Hydrology Study of Upper Little River Watershed, University of Georgia; U.S. Department of Agriculture, Tifton, 1967.
6. W.J. Rawls, P. Yates, and L.E. Asmussen. Calibration of Selected Infiltration Equations for the Georgia Coastal Plain. Report ARS-S-113. Agricultural Research Service, U.S. Department of Agriculture, 1976, 110 pp.
7. J.W. Calhoun. Soil Survey of Tift County, Georgia. Soil Conservation Service, U.S. Department of Agriculture, 1983, 102 pp.
8. R.R. Lowrance, J.K. Sharpe, and J.M. Sheridan. Long Term Sediment Deposition in the Riparian Zone of an Agricultural Watershed. Tech. Rept. ERC 03-84, U.S. Department of Interior Project A-109-GA. U.S. Department of Interior, 1984.
9. J.C. Ritchie, C.A. Ritchie, and G. Plummer. Final Report of Hydrology Study of Upper Little River Watershed. University of Georgia; U.S. Department of Agriculture, Tifton, 1969.
10. C.H. Wharton. The Natural Environments of Georgia. Georgia Department of Natural Resources, Atlanta, 1978, 278 pp.

11. J.M. Sheridan, W.G. Knisel, T.K. Woody, and L.E. Asmussen. Seasonal Variation in Rainfall and Rainfall-Deficient Periods in the Southern Coastal Plan and Flatwoods Regions of Georgia. Bull. 243. College of Agriculture Research, University of Georgia, Athens, 1979, 73 pp.
12. W.M. Snyder and L.E. Asmussen. Subsurface Hydrograph Analysis by Convolution. Journal of Irrigation and Drainage Division, ASCE, Vol. 98, No. IR3, 1972, pp. 405-418.
13. R.K. Hubbard and J.M. Sheridan. Water and Nitrate Losses from a Small Upland Coastal Plain Watershed. Journal of Environmental Quality, Dec. 1983, pp. 291-295.
14. W.G. Knisel and J.M. Sheridan. Procedure for Characterizing Hydrologic Processes in the Coastal Plain of the Southeastern United States. In Proc., International Symposium on Hydrology of Large Flatland Areas, Buenos Aires, Argentina, 1983.
15. A. Shirmohammadi, W.G. Knisel, and J.M. Sheridan. An Approximate Method for Partitioning Daily Streamflow Data. Journal of Hydrology, Vol. 74, No. 3/4, 1984, pp. 335-354.
16. J.C. Ritchie, C.A. Ritchie, and G.L. Plummer. Plant Ecology Studies on the Little River Watershed: Monthly Soil Moisture Variations. In Progress Report of Hydrology Study of Upper Little River Watershed. University of Georgia; U.S. Department of Agriculture, Tifton, 1968.
17. W.O. Ree and W.R. Gwinn. The Virginia V-Notch Weir. Report ARS-41-10. Agricultural Research Service, U.S. Department of Agriculture, 1959, 7 pp.
18. P. Yates. Design, Construction, and Operation of Streamflow-Measuring Facilities in the Little River Watershed, Georgia. Report ARS-S-148. Agricultural Research Science, U.S. Department of Agricultural, 1976, 15 pp.
19. P. Yates. V-Notch Weir for Submerged Flow Measurement. Agricultural Engineering, Vol. 51, No. 3, 1970, pp. 132-133.
20. P. Yates. Research Program and Facilities of the Southeast Watershed Research Program. Lab Note SEWRL 047801. Southeast Watershed Research Laboratory, Tifton, Ga., 1978, 52 pp.
21. P. Yates and J.M. Sheridan. Flow Measurement of Low-Gradient Streams in Sandy Soils. In Proc., International Symposium on Hydrometry, Koblenz, Germany, Vol. 1, United Nations Education, Scientific and Cultural Organization--World Meteorological Organization--International Association of Hydrological Sciences, 1973.
22. W.C. Mills, J.M. Sheridan, and V.A. Ferreira. Hydrologic Measurements on Southern Coastal Plain Experimental Watersheds. Paper 84-2006. Presented at Summer Meeting of the American Society of Associated Executives, Knoxville, Tenn., June 24-27, 1984.
23. J.M. Sheridan, P. Yates, and L.E. Asmussen. Hydrology, Nutrient, and Sediment Movement from Southeastern Coastal Plain Watersheds. In Proc., International Symposium on Nutrient Cycling in Agricultural Ecosystems. Special Publication 23. College of Agriculture, Univ. of Georgia, Athens, 1983.
24. T.C. Wei and J.L. McGuiness. Reciprocal Distance Squared Method: A Computer Technique for Estimating Areal Precipitation. Report ARS-NC-8. Agricultural Research Service, U.S. Department of Agriculture, 1973.
25. Guidelines for Determining Flood Flow Frequency. Bulletin 17B. U.S. Water Resources Council, 1981.
26. H.G. Golden and M. Price. Flood Frequency Analysis for Small Natural Streams in Georgia. Report 76-0511. U.S. Geological Survey, 1976, 91 pp.
27. M. Price. Floods in Georgia, Magnitude and Frequency. Water-Resource Investigations 78. U.S. Geological Survey, 1979, 137 pp.
28. S.H. McCrory, D.G. Baxter, D.L. Yarnell, L.A. Jones, and W.J. Schlick. Report Upon the Cypress Creek Drainage District, Desha and Chicot Counties, Arkansas. Bull. 198. Office of Experiment Stations, U.S. Department of Agriculture, 1915.
29. J.C. Stephens and W.C. Mills. Using the Cypress Creek Formula to Estimate Runoff Rates in the Southern Coastal Plain and Adjacent Flatwoods Land Resource Areas. Report ARS-41-95. Agricultural Research Service, U.S. Department of Agriculture, 1965, 17 pp.
30. Drainage of Agricultural Land: A Practical Handbook for the Planning, Design, Construction, and Maintenance of Agricultural Drainage Systems. Soil Conservation Service, U.S. Department of Agriculture, 1973, 430 pp.
31. Rainfall Frequency Atlas of the United States. Technical Paper 40. U.S. Department of Commerce, 1961, 115 pp.
32. SCS National Engineering Handbook, Section 4: Hydrology. Soil Conservation Service, U.S. Department of Agriculture, 1972, 548 pp.

Culvert Slope and Shape Effects on Outlet Scour

STEVEN R. ABT, CHARLES A. DONNELL, JAMES F. RUFF, and FREDERICK K. DOEHRING

ABSTRACT

Contained in this report are results of a flume study that was conducted to evaluate the effects culvert shape and slope have on outlet scour after 316 min of testing. A circular culvert was tested at 0, 2, and 5 percent slopes. The scour hole characteristics of depth, width, length, and volume were correlated to the discharge intensity ($Qg^{-0.5}D^{-2.5}$) for each slope. The results indicated that an increase in slope subsequently increased the dimensions of scour. The culvert slope significantly affected the scour volume estimates based on prediction equations currently in practice. Tests of circular, square, arch, and rectangular culverts were made with full flow for 316 min. The maximum depth, width, length, and volume of scour were correlated to a modified discharge intensity ($QAg^{0.5}D^{-0.5}$) for each culvert shape. Relationships were derived for predicting outlet scour for each culvert shape. Composite representations were compiled that correlate the dimensions of scour to the modified discharge intensity independent of culvert shape. The results indicate that culvert shape has a limited effect on outlet scour.

One of the major considerations in the design, construction, and rehabilitation of the national highway system is the installation of a properly designed culvert to convey tributary runoff. Culvert design characteristics of shape and installation slope have been key elements of the design process in evaluating the hydraulic efficiency of the runoff conveyance system. However, the effect that culvert shape and installation slope have on culvert outlet scour has thus far not been considered in the design process.

A procedure for predicting scour hole geometry at culvert outlets has been presented by the U.S. Department of Transportation (1). The prediction equations were derived from tests with circular-shaped pipes in a horizontal posture. Unfortunately, the existing prediction procedures for culvert scour do not incorporate a means for adjusting the dimensions of scour for culvert shape or installation slope. Current design procedures require that scour hole geometry be computed independent of culvert shape or slope. Although a single scour computation is convenient for the designer, the culvert shape and slope may significantly affect the scour geometry.

It is the objective of this study to investigate how culvert shape and installation slope affect the scour hole depth, width, length, and volume. Furthermore, design criteria will be presented for estimating localized scour caused by culvert shape and slope. A series of flume tests were conducted at the Engineering Research Center at Colorado State University. Circular, square, arch, and rectangular culverts were tested and the resulting scour holes were documented. Also, circular-shaped culverts were tested with slopes of 0, 2, and 5 percent. The effects of the culvert shapes and slopes are reported herein.

BACKGROUND

The effects of shape and slope on scour holes produced at culvert outlets have not been specifically addressed in past studies. The Bohan (2) and subsequently Fletcher and Grace (3) studies formulated some of the first scour prediction procedures in which the depth, width, length, and volume of scour is depicted as a function of the parameter $Q/D^{2.5}$ where Q is the discharge in cubic feet per second and D is the pipe diameter in feet. Bohan checked the effect of culvert shape on scour hole geometry by passing a single discharge of 0.087 ft^3/sec through circular, square, rectangular, and arch culverts. Each culvert was fabricated with the same cross-sectional area of 0.87 ft^2. Bohan concluded that for both minimum and maximum tailwater (TW) conditions, the culvert shape had little effect on the scour hole geometry.

Fletcher and Grace recognized that for culvert shapes other than circular, the parameter $Q/D^{2.5}$ should be adjusted by a coefficient based on the Froude number at the culvert outlet. However, because the coefficient was based on the flow parameters, the scour geometry was not directly correlated to pipe shapes.

Grace (4) presented a series of equations to estimate the length of a riprap blanket required to protect and reduce scour hole size. He formulated a prediction procedure in which the length of protection is a function of the culvert shape, tailwater depth, discharge, and culvert diameter. Grace presented the design criteria:

$TW < 0.5\ D_o$

Circular and Square Outlets

$$L_{sp}/D_o = 1.8\,[Q/D_o^{5/2}] + 7 \qquad (1)$$

Rectangular and Other Outlets

$$L_{sp}/D_o = 1.8\,[q/D_o^{3/2}] + 7 \qquad (2)$$

$TW \geqq 0.5\ D_o$

Circular and Square Outlets

$$L_{sp}/D_o = 3.0\,[Q/D_o^{5/2}] \quad (3)$$

Rectangular and Other Culverts

$$L_{sp}/D_o = 3.0\,[q/D_o^{3/2}] \quad (4)$$

where

L_{sp} = the length of the stone protection (ft);
Q = the discharge (ft^3/sec);
q = the discharge per foot of outlet width; and
D_o = the diameter of the circular culvert, width and height of a square culvert, and height of a rectangular culvert (ft).

Similar expressions were presented for channel revetment and cellular blocks. Grace recognized that the shape of the culvert outlet affects the geometry of scour and should be addressed in the design procedure.

Ruff et al. (5) and Mendoza (6) performed an investigation in which they studied how square culvert scour geometry compared with scour from a circular culvert. The Froude number was correlated to the dimensionless parameters of depth (d_{sm}/R_H), length L_{sm}/R_H), and volume (V_{sm}/R_H^3) where R_H is the hydraulic radius. The results indicated that a circular culvert yielded a more conservative volume of scour than a square culvert when the diameter equaled the culvert height. However, comparisons of the depth of scour and length of scour yielded similar results independent of the culvert shape for Froude numbers ranging from 2 to 6.5.

Chen (7) suggested that under conditions of equivalent discharge, a square culvert with height equal to the diameter of a circular culvert would reduce scour on a mild slope. However, he did not provide any guidelines on how scour was reduced or on the extent of the reduction.

On the basis of this limited information base, it is apparent that different culvert shapes may affect scour geometry at culvert outlets. It is the purpose of this investigation to indicate the relative differences in scour that may be attributed to culvert shape. Furthermore, the effects of culvert installation slope on outlet scour will be examined.

FACILITY AND BED MATERIAL

Scour tests were performed in a flume 8 ft deep, 20 ft wide, and 100 ft long. The operational facility is shown in Figure 1.

The culverts were projected 7.2 ft into the flume parallel to the sidewalls; to minimize headwall effects, to allow for fully developed flow, and to maintain consistency with previous investigations. Circular, square, arch, and rectangular shapes were used (Figure 2). Table 1 summarizes the dimensions of each culvert shape. The rectangular shape is based on a 1.5 horizontal to 1.0 vertical ratio. The culverts were constructed so that the characteristic height of each was 4.0 in.

A 4-in. circular steel pipe was used to determine the culvert slope effects on outlet scour. The pipe was installed as presented in Figure 3 to enhance pipe slope modifications. Pipe slopes of 0, 2, and 5 percent were tested.

The flume was filled with a uniformly graded sand to the culvert invert. The soil properties were median grain diameter (d_{50}), 0.0061 ft; standard deviation [$\sigma = (d_{84}/d_{16})^{0.5}$], 1.33; unit weight ($\gamma$), 93.8

FIGURE 1 Test facility.

FIGURE 2 Model culverts.

TABLE 1 Culvert Shape Dimensions

Culvert Shape	Cross-sectional Area (ft^2)	Hydraulic Radius (ft)	Maximum Width (ft)	Maximum Height (ft)
Circular	0.087	0.083	0.33	0.33
Square	0.111	0.083	0.33	0.33
Arch	0.143	0.101	0.50	0.33
Rectangular	0.167	0.100	0.52	0.33

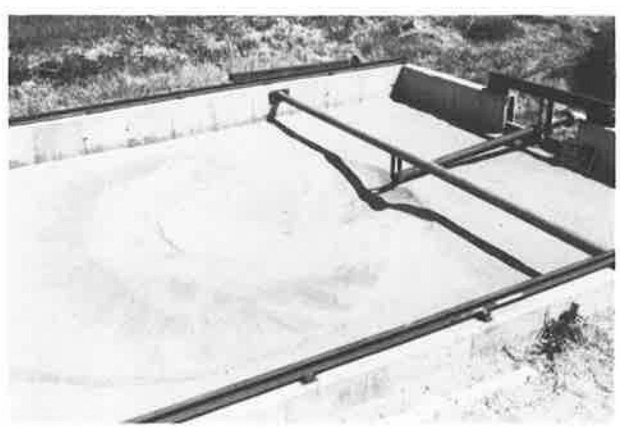


FIGURE 3 Model test stand for evaluating scour from sloping culverts.

lb/ft^3; fall velocity (ω), 0.89 ft/sec; and angle of repose (ϕ), 34.8 degrees. The sand bed material properties were determined in accordance with ASTM procedures.

This particular sand was used in deriving many of the scour hole prediction procedures currently used by the U.S. Department of Transportation. Although this sand is seldom found naturally, continuity between laboratory testing programs and the comparability of results was determined to be advantageous.

Scour hole contours were taken from a motorized carriage that rested on rails on top of the flume's sidewalls. A small motorized cart, which housed a point gauge, was mounted on the carriage; this allowed data acquisition at any point in the flume. The point gauge resolution was 0.01 ft.

TEST PROCEDURE

The bed was leveled adjacent to the culvert invert elevation. Water was pumped into the flume until the surface reached a tailwater elevation of approximately 0.45 times the height of the culvert above the invert. Once the desired tailwater elevation had been reached, the culvert control valve was opened to permit flow at the desired discharge. Discharges ranged from 0.36 to 2.09 ft^3/sec. (A summary of the culvert shapes, discharges, and discharge intensities is presented in Table 2.) The culverts flowed full during all testing conditions. The scour holes were contoured after 31, 100, and 316 min of testing. These times were selected for consistency and comparison with previous outlet scour testing studies.

TABLE 2 Summary of Culvert Shapes, Discharges, and Discharge Intensities

Pipe Shape	Slope %	Discharge cfs	Discharge Intensity $Qg^{-0.5}D^{-2.5}$	Modified Discharge Intensity $QA^{-1}(gR_H)^{-0.5}$
Circular	0	0.37	1.0	2.53
	0	0.55	1.5	2.87
	0	0.74	2.0	5.13
	0	0.92	2.5	6.40
	0	1.14	3.13	8.02
	0	0.36	0.98	2.50
	0	0.86	2.36	6.0
	2	0.37	1.0	--
	2	0.55	1.5	--
	2	0.74	2.0	--
	2	0.92	2.5	--
	5	0.37	1.0	--
	5	0.55	1.5	--
	5	0.74	2.0	--
	5	0.92	2.5	--
Square	0	0.46	--	2.50
		0.55	--	3.0
		0.73	--	3.0
		1.09	--	6.0
Arch	0	0.61	--	2.36
		0.75	--	3.00
		0.97	--	3.76
		1.18	--	4.7
		1.45	--	5.63
Rectangular	0	0.75	--	2.50
		0.90	--	3.00
		1.20	--	4.00
		1.50	--	5.00
		1.73	--	5.77
		2.09	--	7.00

RESULTS AND DISCUSSION

Culvert Slope Analysis

The tests evaluating the scour from sloped culverts were conducted using only a circular pipe with slopes of 0, 2, and 5 percent. The data analysis was conducted in a manner similar to the procedures for formulating the design criteria presented by the U.S. Department of Transportation (1) for comparison of the results. The maximum scour hole dimensions of depth (d_{sm}), width (W_{sm}), length (L_{sm}), and volume (V_{sm}) were correlated to the discharge, culvert diameter, and culvert slope. The hole characteristic dimensions were expressed as dimensionless parameters of d_{sm}/D, W_{sm}/D, L_{sm}/D, and V_{sm}/D^3 to represent the depth, width, length, and volume of scour, respectively.

Graphic representations were compiled that correlate the dimensionless depth, width, length, and volume of scour to the discharge intensity (DI) for each slope. For this analysis, the DI is defined as

$$DI = Q/g^{0.5}D^{2.5} \tag{5}$$

where

- Q = the discharge (ft^3/sec),
- g = the acceleration of gravity (ft/sec^2), and
- D = the culvert diameter (ft).

The resulting logarithmic plots relating the scour hole dimensions to the discharge intensities are presented in Figures 4 through 7. It is important to note that in all cases, little difference could be delineated between data points representing each slope. Therefore, data were consolidated to formulate a single relation where appropriate.

A power regression line was fit through each logarithmic plot; this yielded a series of expressions of the following general form:

$$y = a\,x^b \tag{6}$$

where

- y = the dependent parameter of d_{sm}/D, W_{sm}/D, L_{sm}/D, or V_{sm}/D^3;
- a = a constant; and
- b = the slope of the linearized plot.

Replacing the independent parameter in Equation 6 with the DI yields the following expression:

$$d_{sm}/D, W_{sm}/D, L_{sm}/D, \text{or } V_{sm}/D^3 = a[Q/(g^{0.5}D^{2.5})]^b \tag{7}$$

A summary of the coefficients for Equation 7 and the regression coefficient, r^2, is presented in Table 3.

It is observed in Figure 4 that the 5 percent sloped culvert yields a slightly greater maximum scour depth than does a 0 to 2 percent sloped culvert. There was no appreciable difference between the scour hole depths of the 0 and 2 percent slopes. The average increase of approximately 10 percent in scour hole depth was observed within the range of DIs of from 1.0 to 2.5.

Examination of Figure 5 indicates that culverts with both 2 and 5 percent slope increase the width of scour compared with those with 0 percent slope. Because of the data distribution, a single line was plotted that defined the width of scour for slopes of 2 through 5 percent. The sloped culverts increased the width of scour by approximately 25 percent.

TABLE 3 Summary of Coefficients for Equation 7

Dependent Parameter	Culvert Slope %	a	b	r^2
d_{sm}/D	5	2.68	0.31	0.81
d_{sm}/D	0 & 2	2.51	0.27	0.59
W_{sm}/D	2 & 5	16.36	0.61	0.73
W_{sm}/D	0	12.43	0.59	0.91
L_{sm}/D	2 & 5	26.18	0.65	0.92
L_{sm}/D	0	18.24	0.93	0.71
V_{sm}/D^3	2 & 5	220.48	1.60	0.98
V_{sm}/D^3	0	162.11	1.57	0.95

Equation 7: Dependent parameter = $a(D.I.)^b$

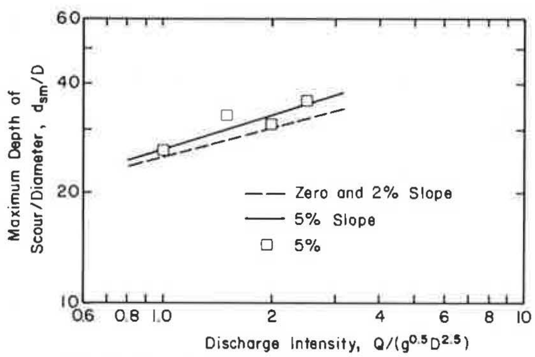


FIGURE 4 Slope influence on scour depth versus discharge intensity.

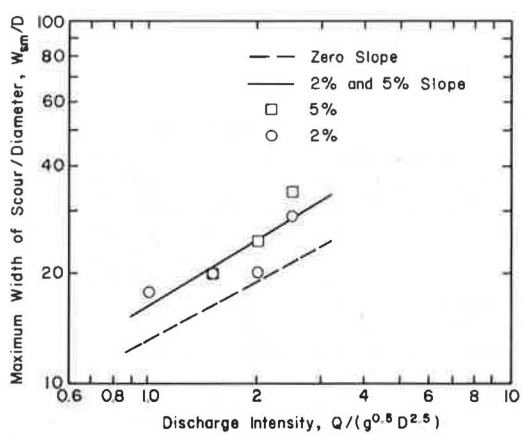


FIGURE 5 Slope influence on scour width versus discharge intensity.

Figure 6 presents the length of scour estimates as related to the slope of the culvert. Similar to Figure 5, the data coincided for the 2 and 5 percent slopes and were thereby consolidated to a single relationship. It is observed that at low DIs (DI < 1.5), the culvert slope significantly increased the length of scour compared with the 0-percent culvert slope. However, as the discharge intensity increases, the prediction lines converge. It is hypothesized that the sloped culvert increases the

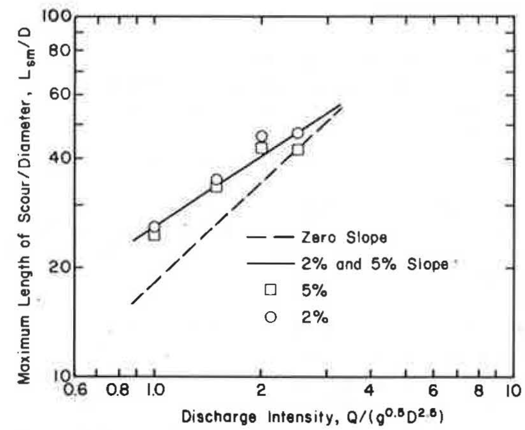


FIGURE 6 Slope influence on scour length versus discharge intensity.

scour length because the outlet jet is concentrated toward the bed. When the culvert is horizontal, a larger portion of the jet is directed away from the bed and the jet energy is dissipated in the tailwater. The sloped culverts increased the length of scour approximately 25 to 30 percent.

The scour volume relationships are shown in Figure 7. The volume of scour increases as the culvert slope increases within the range of slopes tested. The increase in scour volume of culverts at 2 and 5 percent slope was approximately 40 percent above that at the 0-percent slope. Therefore, the culvert slope significantly affects the scour volume estimates based on prediction equations that are in current practice.

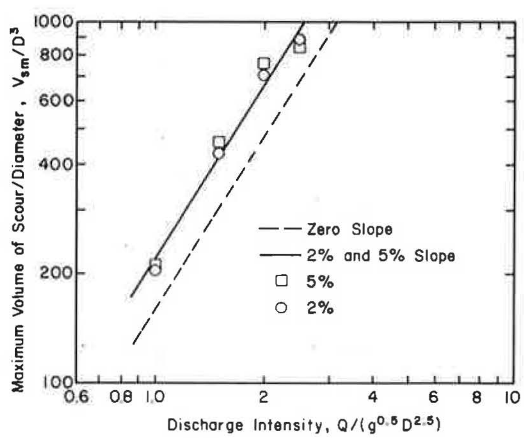


FIGURE 7 Slope influence on scour volume versus discharge intensity.

Culvert Shape Analysis

The procedure presented for performing the culvert slope analysis is applicable only for circular-shaped culverts because the culvert diameter is not characteristic of square, rectangular, or arch culverts (8). Therefore, the DI presented in Equation 5, must be modified to account for the varying flow geometries. The modified discharge intensity (DI*) was defined as

$$DI^* = Q/A(gR_H)^{0.5} \quad (8)$$

where

Q = the discharge (ft^3/sec),
A = the cross-sectional area of flow (ft^2),
g = the gravitational acceleration (ft/sec^2), and
R_H = the culvert hydraulic radius (ft).

The modified discharge intensity remains a dimensionless parameter that is a function of the discharge and culvert shape. The $Ag^{-0.5}R_H^{-0.5}$ term is the shape factor that uniquely reflects the different culvert shapes.

The scour hole characteristics of depth, width, length, and volume are expressed in the dimensionless form as d_{sm}/R_H, W_{sm}/R_H, L_{sm}/R_H, and V_{sm}/R_H, respectively. These relationships are based on the maximum scour hole characteristic dimensions generally obtained after 316 min of testing. Again, a 316-min testing duration was selected for consistency and a comparison with prior experimentation. A graphic representation correlating each dimensionless scour hole parameter to the modified discharge intensity (DI*) was compiled for each culvert shape. The dimensions of scour versus modified discharge intensity are presented in Figure 8 for the arch-shaped culvert. Similar plots were compiled for the

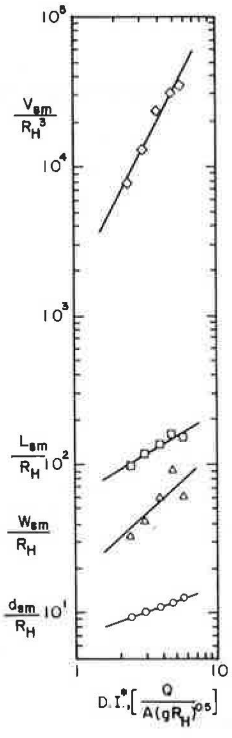


FIGURE 8 Dimensions of scour versus the modified discharge intensity for the arch culvert.

circular, square, and rectangular culvert shapes. A power regression line was fit through each logarithmic plot yielding a series of expressions of the following general form:

$$d_{sm}/R_H, W_{sm}/R_H, L_{sm}/R_H, V_{sm}/R_H^3 = c[Q/(Ag^{0.5}R_H^{0.5})]^d \quad (9)$$

where c is a constant and d is the slope of the linearized plot.

Table 4 summarizes the coefficients that are applicable to Equation 9. It is evident that a series of expressions can be formulated correlating the scour hole characteristics to the modified discharge intensity for a particular noncircular culvert shape.

Regression coefficients tabulated in Table 4 were compared for similar scour parameters for each culvert shape. It is observed that the c and d coefficients depicting the regression constant and slope, respectively, indicate a considerable variation in each of the predictive relationships. Figure 9 presents the regression lines that predict the scour volume for circular, square, rectangular, and arch culverts. Similar plots were compiled for the depth, width, and length of scour. It is evident that the results did not indicate that any particular culvert shape was advantageous when outlet scour is considered.

TABLE 4 Summary of Coefficients for Equation 9

Culvert Shape	Dependent Parameter	Independent Parameter	c Coefficient	d Coefficient
Circular	d_{sm}/R_H	D.I.*	7.59	0.29
	W_{sm}/R_H	D.I.*	28.98	0.58
	L_{sm}/R_H	D.I.*	31.05	0.92
	V_{sm}/R_H^3	D.I.*	2,460.13	1.56
Square	d_{sm}/R_H	D.I.*	9.26	0.20
	W_{sm}/R_H	D.I.*	47.69	0.24
	L_{sm}/R_H	D.I.*	43.86	0.93
	V_{sm}/R_H^3	D.I.*	2,646.71	1.80
Rectangular (1.5H:1.0V)	d_{sm}/R_H	D.I.*	7.83	0.73
	W_{sm}/R_H	D.I.*	21.93	0.73
	L_{sm}/R_H	D.I.*	76.47	0.48
	V_{sm}/R_H^3	D.I.*	4,899.35	1.23
Arch	d_{sm}/R_H	D.I.*	7.16	0.32
	W_{sm}/R_H	D.I.*	17.73	0.89
	L_{sm}/R_H	D.I.*	62.91	0.55
	V_{sm}/R_H^3	D.I.*	1,788.65	1.80

Note: Dependent Parameter = $c(DI^*)^d$.

A series of composite logarithmic graphic representations were compiled that correlate the scour hole depth, width, length, and volume parameters to the modified discharge intensity as presented in Figures 10 through 13, respectively. Each plot portrays the scour hole data collected for the circular, square, rectangular, and arch-shaped culverts. A power regression line was fit to each linearized plot, yielding a series of general expressions of the form presented in Equation 9. A summary of the coefficients for the consolidated data is presented in Table 5. An examination of Figures 10 through 13 indicates that the consolidated data correlate well with the modified discharge intensity. It is difficult to observe any unique trends of a particular culvert shape in the consolidated plots. The data indicate that the culvert shape has little effect on the dimensions of outlet scour.

CONCLUSIONS

A series of relationships have been presented for predicting the dimensions of scour for culvert slopes of 0 to 5 percent as a function of the DI. It is apparent that the culvert slope affects the dimensions of scour at culvert outlets. In all cases, the dimensions of scour increase as the DI in-

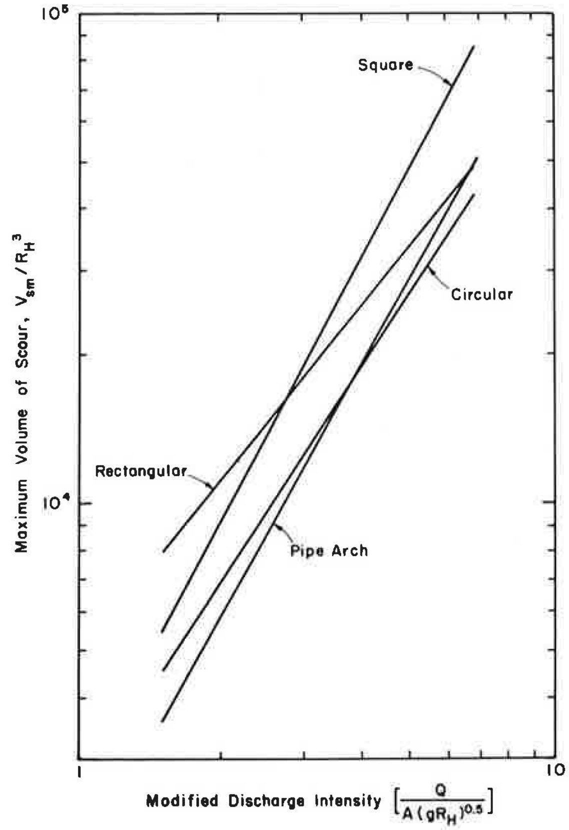


FIGURE 9 The volume of scour versus the modified discharge intensity for the circular, square, arch, and rectangular culverts.

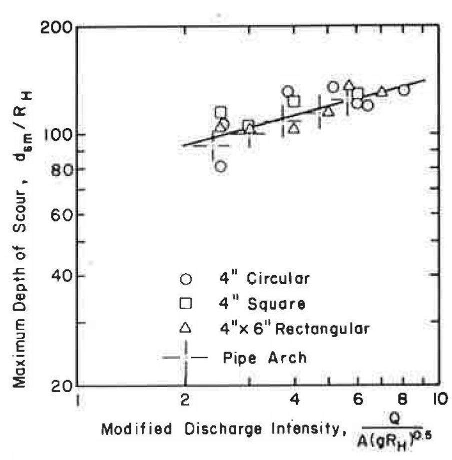


FIGURE 10 Depth of scour versus modified discharge intensity for the consolidated culvert shapes.

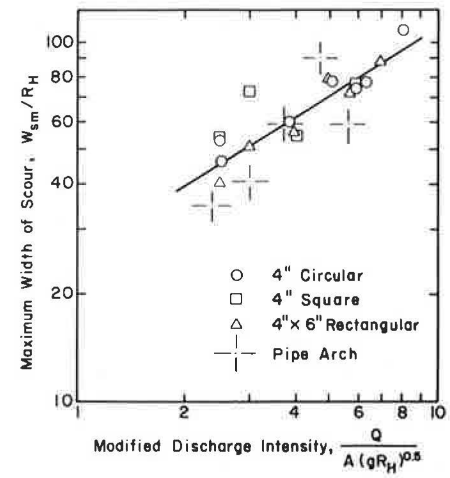


FIGURE 11 Width of scour versus modified discharge intensity for the consolidated culvert shapes.

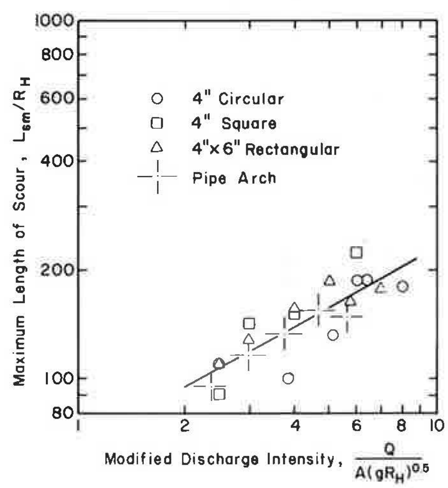


FIGURE 12 Length of scour versus modified discharge intensity for the consolidated culvert shapes.

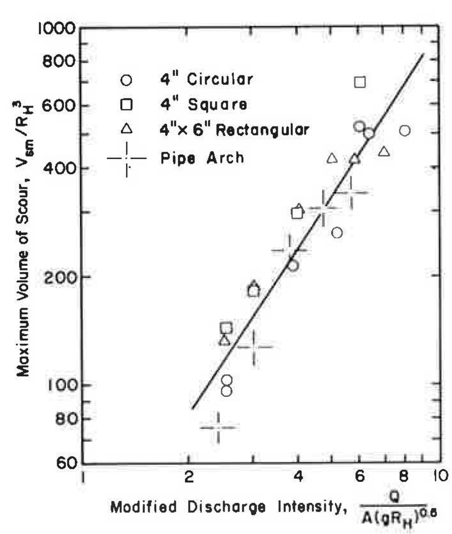


FIGURE 13 Volume of scour versus modified discharge intensity for the consolidated culvert shapes.

TABLE 5 Summary of Coefficients for Consolidated Data Applied to Equation 9

Dependent Parameter	Independent Parameter	c	d	r^2
d_{sm}/R_H	D.I.*	7.96	0.26	0.62
W_{sm}/R_H	D.I.*	26.42	0.62	0.70
L_{sm}/R_H	D.I.*	64.54	0.56	0.71
V_{sm}/R_H^3	D.I.*	3000.60	1.51	0.88

Note: Dependent Parameter = $c(DI^*)^d$.

creases. The culvert slope significantly affects the scour estimates based on prediction equations that are in current practice.

Relationships are derived for predicting the dimensions of outlet scour for circular, square, rectangular, and arch culverts in a horizontal posture. A series of composite representations have been compiled relating the depth, width, length, and volume of scour to the modified DI. The composite plots indicate that the culvert shape has little effect on the dimensions of outlet scour.

ACKNOWLEDGMENTS

The authors wish to thank the U.S. Army for funding the work of Donnell during this research, and the Colorado State Agricultural Experiment Station for funding the investigation. Zane Short and Robert Brodin are acknowledged for assisting in the data acquisition and reduction.

REFERENCES

1. Hydraulic Design of Energy Dissipators for Culverts and Channels. Chapter V, Hydraulic Engineering Circular 14, Office of Engineering, FHWA, U.S. Department of Transportation, Sept. 1983.
2. J.P. Bohan. Erosion and Riprap Requirements at Culvert and Storm-Drain Outlets. Report H-70-2. U.S. Army Corps of Engineers Waterways Experiment Station, Vicksburg, Miss., 1970, 53 pp.
3. B.P. Fletcher and J.L. Grace, Jr. Practical Guidance for Estimating and Controlling Erosion at Culvert Outlets. Report H-72-50. U.S. Army Corps of Engineers Waterways Experiment Station, Vicksburg, Miss., 1972, 43 pp.
4. J.L. Grace, Jr. Erosion Control at Culvert Outlets. Proc., 1980 Annual Meeting of the American Society of Civil Engineers, Hollywood, Fla., Oct. 1980, pp. 1-14.
5. J.F. Ruff, S.R. Abt, C. Mendoza, A. Shaikh, and R. Kloberdanz. Scour at Culvert Outlets in Mixed Bed Materials. Office of Research and Development, FHWA, U.S. Department of Transportation, 1981, 115 pp.
6. C. Mendoza. Headwall Influence on Scour at Culvert Outlets. M.S. thesis. Colorado State University, Ft. Collins, Colo., 1980.
7. Y.H. Chen. Scour at Outlets at Box Culverts. M.S. thesis. Colorado State University, Ft. Collins, Colo., 1970.
8. C.A. Donnell and S.R. Abt. Culvert Shape Effects of Localized Scour. Hydrographic Engineering Publication CER82-83CAD-SRA42. Colorado State University, Ft. Collins, Colo., 1983.

Stormwater Management Detention Pond Design Within Floodplain Areas

PAUL H. SMITH and JACK S. COOK

ABSTRACT

A unique approach to stormwater management for projects requiring mitigation of additional runoff caused by increases in paved surface areas is presented in this paper. Based on a design project developed for the General Foods Corporate Headquarters site in Rye, New York, a stormwater detention pond has been implemented within the floodplain of an adjacent water course. Encroachment of construction activities within a floodplain required the development of a detention pond that was capable of controlling excess runoff from adjacent areas while providing continued floodplain storage volume capacity. This methodology minimized the impacts of flooding on adjacent properties and provided suitable land areas for development in accordance with the intended use of the property. Occurrence of peak flooding along the watercourse did not coincide with peak stormwater runoff conditions from the smaller adjacent drainage area. By utilizing flood hydrograph principles and analyses that were developed by the Soil Conservation Service, U.S. Department of Agriculture, it was possible to develop a detention pond to provide a stormwater management phase and a flood control phase. Computerized analyses were compared for pre- and postdevelopment conditions using stormwater runoff and flood flow data on the basis of storms with return period frequencies of 10, 25, 50, and 100 years. By providing inlet pipes and outlet structures to control detention pond storage, peak flows from the pond to the watercourse and peak flood flows on the watercourse were reduced. The detention pond provides an aesthetic and effective method of mitigating flooding impacts that might have resulted from site development.

Continuing growth in urban areas coupled with increasing land values and decreasing availability of suitable development sites often causes federal, state, and local governments and private property owners to seek unique and innovative ways to pursue development. The use of detention ponds as a means of stormwater management and runoff control has become a widely accepted method for controlling stormwater runoff from highways, roadways, and new land use developments.

Construction of a detention pond within a floodplain provides a method for controlling adjacent site runoff while providing continued floodplain storage volume capacity. As a result, substantial benefit can be derived by adjacent property owners upstream and downstream of a development site by using such a detention pond to attenuate peak flood flows on the watercourse.

This approach to stormwater management is applicable to all projects that require mitigation of impacts that result from additional runoff caused by increases in paved surface areas. Transportation facilities, including new highways and interchanges; roadway widening and realignments; airport expansions; and construction of structures and parking facilities for intermodal transfer, maintenance, storage, and related facilities can all benefit from improved and alternative methods of runoff control and stormwater management.

The term "detention pond," as used in this discussion, refers to a man-made depression that will retain water year-round and provide for temporary storage and controlled discharge of excess water through an outlet structure. When a detention pond is to be constructed within the floodplain of a watercourse, additional measures must be developed to maintain the existing flood storage capacity of the floodplain while permitting increased storage capacity for detention and control of excess stormwater runoff from adjacent areas.

The General Foods Corporation has successfully implemented such a development program for their corporate headquarters building and surrounding access roads located in Rye, New York. Completion of this project required development of a stormwater detention pond approximately 6.3 acres in surface area, primarily located within the 100-yr floodplain of Blind Brook. Figure 1 shows the extent of encroachment of the 100-yr floodplain on the General Foods headquarters site before development. Design and construction of the detention pond had to be performed using the following criteria:

1. Control of postdevelopment rates of stormwater runoff tributary to the pond at or below predevelopment peak discharge rates to Blind Brook during the 10-, 25-, 50-, and 100-yr return period storms; and
2. Replacement of all floodplain storage volume that has been removed because of construction of buildings, embankments (berms), or other structures within the existing 100-yr floodplain limits.

To maintain preconstruction floodplain storage volume capacity within the site, it was necessary to divert a portion of the floodwaters from Blind Brook into the detention pond. Construction within the flood areas could not result in any measurable increase in water surface elevations along Blind Brook at any point upstream or downstream from the site at the Westchester Avenue box culvert and the Bowman Avenue Bridge.

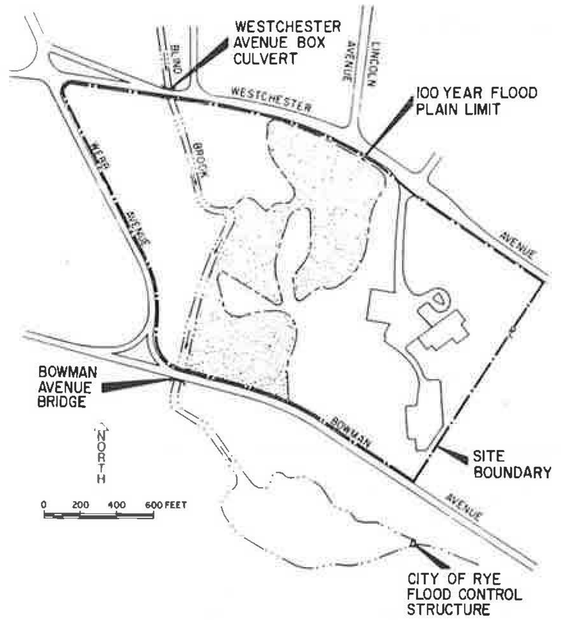


FIGURE 1 Predevelopment conditions.

Preliminary investigations indicated that the detention pond could be designed to provide a stormwater management phase and a flood control phase as an acceptable approach when flood hydrograph principles and analyses were considered. The goal of the first phase is to limit peak site runoff that results from increased impervious areas and to handle off-site runoff generated from tributary areas adjacent to the site. The detention pond location, shown in Figure 2, has a total drainage area of 89.2 acres. Because site flooding from Blind Brook resulted from a considerably larger drainage area of approximately 4,060 acres, hydrologic principles dictate that peak stormwater runoff conditions will not occur simultaneously. Peak runoff discharge from the immediate drainage area will occur before peak flood conditions within the Blind Brook floodplain.

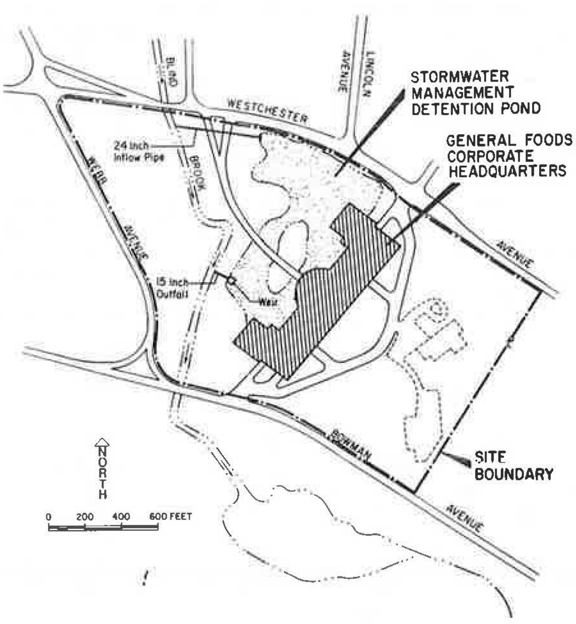


FIGURE 2 Postdevelopment conditions.

During the second phase of detention pond operation, partial diversion of flood water from Blind Brook to the pond is accomplished through an appropriately designed inflow pipe. The amount of flood water diversion from Blind Brook to the pond is based on the floodplain storage volumes that are lost as a result of the encroachment of development. Although these two phases can be considered essentially distinct, they must occur simultaneously during a design storm event.

The design approach is to provide a detention pond with an outlet structure that is sufficient to substantially reduce flood flows that result from the small watershed immediately adjacent to the site, and simultaneously to attain an established pond water surface elevation that corresponds to the storage volume to be returned to the floodplain. In this way, as inundation of the detention pond by Blind Brook floodwater occurs, sufficient detention pond storage volume is also readily available to meet the previously outlined criteria.

Verification of the feasibility of the foregoing approach was performed using conservative empirical approaches as outlined by the Soil Conservation Service (SCS), U.S. Department of Agriculture (1-3). These design approaches have their bases in the unit hydrograph theory. Flood routing (4) was extremely useful in providing reasonable estimates of expected peak flows and their time of occurrence. In addition, these methods provided good approximation of retardant pool storage, which was required within the detention pond, as well as the approximate size of the required outlet structure.

EXISTING AND FUTURE FLOOD ELEVATIONS

Water surface profiles for Blind Brook for pre- and postdevelopment conditions were prepared by using the Hydrologic Engineering Center (HEC-2) model (5). Input to the HEC-2 computer program consists of cross sections of typical channel segments, other parameters describing flood conditions such as channel roughness and expansion and contraction coefficients, bridge and culvert data, and peak flow data.

The analyses utilized cross-sectional data obtained from the Federal Insurance Administration (FIA) (6) study performed for the town of Rye in 1978. Cross sections of the channel within the site were determined from topographic contour mapping of the site to more accurately reflect channel conditions and more fully describe flood zone areas.

Input parameters generated by the FIA study to describe the Westchester Avenue culverts were utilized. The city of Rye flood control structure was described in data obtained through field surveys. To preserve the accuracy of the calibrated FIA model, the channel roughness and contraction and expansion coefficients determined by the FIA were utilized in the computer runs performed for this project.

Flood flows for the 10-, 50-, and 100-yr events used by the FIA were also utilized in this study. These peak flows are based on SCS flood routing for Blind Brook and were adjusted according to existing stream gauge information and frequency analyses. Starting water surface elevations downstream of the structure were also taken from the FIA study.

The 25-yr flood peak was approximated using a storm frequency (semilogarithmic) plot as shown in Figure 3. The initial elevation for the 25-yr condition was established using a semilogarithmic plot as shown in Figure 4.

Future conditions were modeled by simulating building encroachments on the existing stream channel cross sections and modifying the cross sections to allow for cut and fill. Flow was restricted to

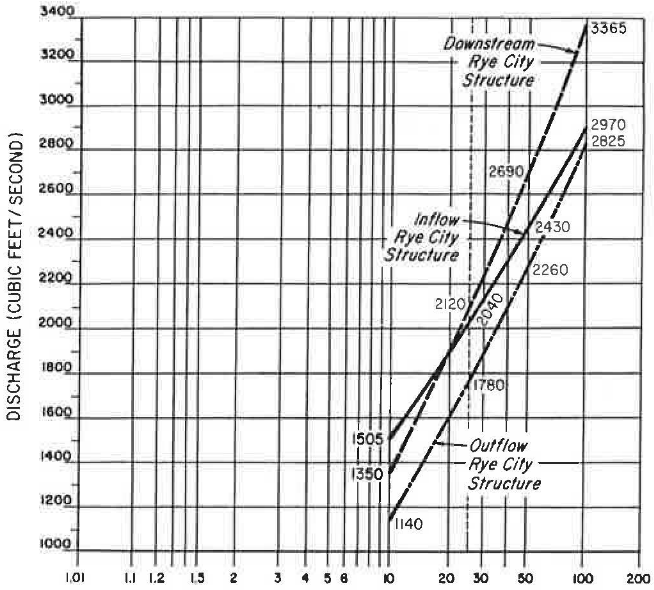


FIGURE 3 Blind Brook flood discharge.

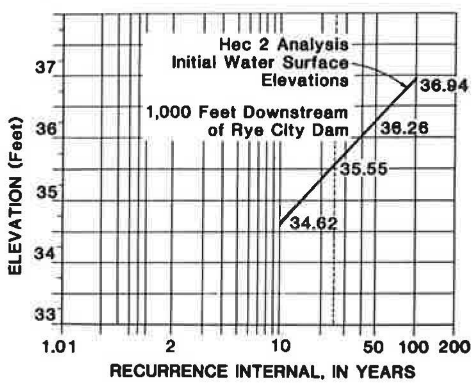


FIGURE 4 Blind Brook flood water surface elevations.

the channel section area outside the pond berm. Improvements in channel conveyance due to grading, grassing, and brush removal were reflected in the HEC-2 input by reduction of the channel roughness coefficient for improved overbank area.

Results of the HEC-2 analyses are summarized in Tables 1 and 2 for pre- and postdevelopment conditions, respectively. These results indicate that no increase in water surface elevations would occur upstream or downstream because of construction of the pond berm or the building.

These analyses verify subcritical flow conditions for this section of Blind Brook. Under subcritical flow, changes in channel conditions at any given location have no hydraulic effect on downstream water surface elevations. Because the detention pond will serve to substantially reduce discharge to Blind Brook and thereby reduce peak flows, conditions downstream of Bowman Avenue were projected to improve slightly after construction of the pond.

ENCROACHMENT VOLUMES

To determine storage volume requirements for the detention pond, several issues had to be considered.

TABLE 1 Blind Brook Water Surface Elevations: Predevelopment Conditions

	Elevation (ft)[a] by Storm Frequency (yr)			
Cross Section	100	50	25	10
1,000 ft downstream of Rye City Dam	36.94	36.26	35.55	34.62
Immediately downstream of Rye City Dam	48.65	47.81	46.74	45.02
Rye City Dam	61.80	61.17	60.54	59.51
120 ft upstream of Rye City Dam	61.96	61.28	60.56	59.46
Bowman Avenue	63.12	62.34	61.70	60.05
225 ft upstream of Bowman Avenue	63.49	62.74	62.12	60.97
1,050 ft upstream of Bowman Avenue	66.84	66.25	65.40	64.79
170 ft downstream of Westchester Avenue	69.26	68.71	68.27	67.50
Westchester Avenue	72.02	71.92	71.52	69.36
1,000 ft upstream of Westchester Avenue	78.96	78.59	77.02	76.00

[a]At mean sea level.

TABLE 2 Blind Brook Water Surface Elevations: Postdevelopment Conditions

	Elevation (ft)[a] by Storm Frequency (yr)			
Cross Section	100	50	25	10
1,000 ft downstream of Rye City Dam	36.94	36.26	35.55	34.62
Immediately downstream of Rye City Dam	48.65	47.81	46.74	45.02
Rye City Dam	61.80	61.17	60.54	59.51
120 ft upstream of Rye City Dam	61.97	61.28	60.56	59.46
Bowman Avenue	63.12	62.34	61.70	60.05
225 ft upstream of Bowman Avenue	63.46	62.71	62.10	60.95
1,050 ft upstream of Bowman Avenue	66.80	66.28	65.54	64.78
170 ft downstream of Westchester Avenue	69.26	68.70	68.24	67.47
Westchester Avenue	72.02	71.91	71.52	69.36
1,000 ft upstream of Westchester Avenue	78.76	78.59	77.02	76.00

[a]At mean sea level.

Pond storage volumes consisted of predevelopment flood plain volumes in overbank areas adjacent to the Blind Brook channel, floodplain volumes displaced by fill material, and additional volumes required because of increased runoff from additional impervious areas within the site. These required volumes are shown in Figure 5.

Volumes of floodplain storage capacity that are affected by design conditions were determined by using average end area calculations for the portion directly affected by proposed construction. These affected volumes included the region confined by the outside toe of the slope of the pond berm and the outside wall of the proposed building. On the basis of predevelopment conditions, floodplain storage volumes were affected by the office building and detention pond construction. A portion of these floodplain volumes was lost directly as a result of construction of earthen embankments and the building. Provisions for returning this volume back to the floodplain were made through excavation within the limits of the pond area. The affected floodplain and encroachment volumes are given in Table 3.

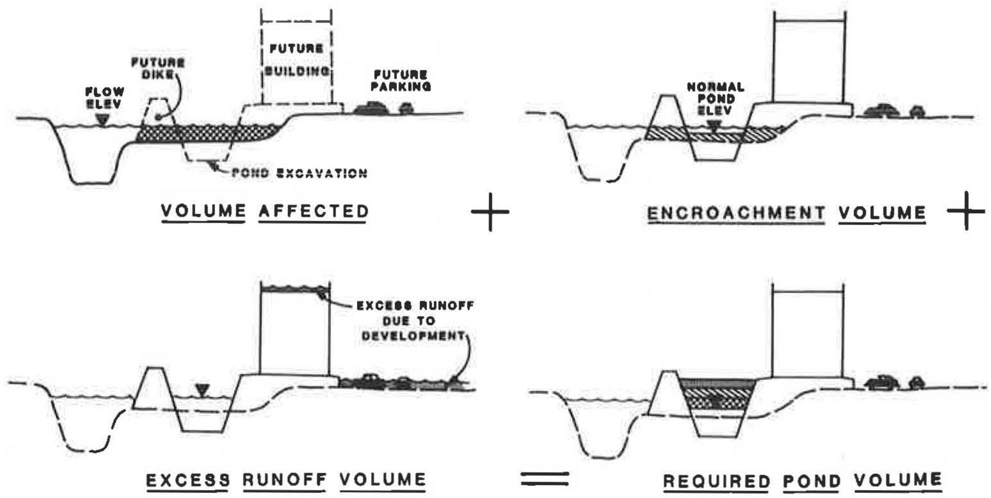


FIGURE 5 Detention pond storage volume.

TABLE 3 Affected Floodplain and Encroachment Volumes

Storm or Flood Frequency (yr)	Affected Floodplain Volume (acre-ft)	Encroachment Volume (acre-ft)
10	10.6	1.4
25	24.0	3.0
50	25.9	4.0
100	26.8	5.6

In addition to flood storage volumes lost because of construction, there was a net increase in runoff volume caused by the increased impervious surface area on the site, which includes parking, road, roof, and pond. Provisions had to be made for storing this excess volume within the pond as well. Calculations of the estimated runoff volume were made with SCS curve numbers (CN) that are based on soil types and land use (see Table 4).

For estimating runoff volumes, the following equation was used to determine the depth of water runoff over the drainage area:

$$Q = (P - 0.2S)^2/(P + 0.8S) \tag{1}$$

where

Q = depth of water discharge (in.),
P = precipitation (in.), and
S = (1,000/CN) - 10.

By using the above equation, the excess runoff volumes for 24-hr duration rainfall amounts over the 37-acre drainage area were estimated and these data are given in Table 5.

Combining flood storage volume losses that result from encroachment and additional runoff caused by development yields storage that must be effectively returned to the floodplain during the design floods. The required pond storage volumes necessary to ensure sufficient runoff detention are given in Table 6.

FLOOD ROUTING

Runoff hydrographs were generated for pre- and post-development site conditions that involve the use of methods outlined by the SCS (1,4). Runoff hydro-

TABLE 4 Retention Structure, Land Use, and Determination of Runoff Curve Number (RCN)

Conditions	Land Use (%)	RCN
Existing		
Undeveloped site (forest and floodplain)	24.2	60
Commercial	7.7	92
Residential (¼-acre lots)	68	75
Weighted curve number		72.6
Rounded weighted curve number		73
Future[a]		
Mixed commercial	24.2	83
Commercial	7.7	92
Residential (¼-acre lots)	68	75
Weighted curve number		78.3
Rounded weighted curve number		78

Note: area = 89.2 acres and soil class B is assumed.
[a]The calculated RCN for development is RCN = [0.64 (98)]/[0.36 (57)] = 83.

TABLE 5 Excess Runoff Volumes from 10-, 25-, 50-, and 100-yr Storms or Floods

Parameter	Storm or Flood Recurrence Frequency (yr) 100	50	25	10
Precipitation, P (in.)	7.2	6.5	5.8	5.0
Predevelopment Curve Number	60.0	60.0	60.0	60.0
Predevelopment Runoff, Q_E (in.)	2.8	2.3	1.8	1.3
Postdevelopment Curve Number	83.0	83.0	83.0	83.0
Postdevelopment Runoff, Q_F (in.)	5.2	4.6	3.9	3.2
Area, A (acres)	37.0	37.0	37.0	37.0
Excess Runoff Volume (acre-ft), $[(Q_F - Q_E)/12]$ A	7.4	7.1	6.5	5.9

TABLE 6 Storm Recurrence Frequency and Volumes

Frequency Occurrence (yr)	(1) Existing Volume (acre-ft)	(2) Total Encroachment Volume (acre-ft)	(1) + (2) Required Pond Volume (acre-ft)
100	26.8	13.0	39.8
50	25.9	11.1	37.0
25	24.0	9.5	33.5
10	10.6	7.3	18.0

graphs provide the engineer with a model of any flood of given duration and return period. The inflow hydrograph is used to perform flood routing through the pond outlet works and spillways and simultaneously allow for storage of inflow runoff volume.

Runoff curve numbers for pre- and postdevelopment conditions were determined on the basis of land use. Inflow hydrographs for 24-hr duration storms were generated using SCS computer program (7). Results of these calculations and computer analyses are given in Table 7.

TABLE 7 Pre- and Postdevelopment Peak Flows

Return Period (yr)	Predevelopment		Postdevelopment	
	Peak Discharge (ft^3/sec)	Time to Peak (hr)	Peak Discharge (ft^3/sec)	Time to Peak (hr)
10	54	10.5	79	10.3
25	72	10.5	101	10.3
50	87	10.5	120	10.3
100	104	10.5	140	10.3

Detention pond design includes a 24-in. diameter reinforced-concrete inflow pipe, which allows water to flow into the pond, thus providing water recirculation through the pond during normal conditions. During flooding conditions, this pipe removes storm flow from Blind Brook, stores these volumes in the pond, and thereby provides the required storage volume that was calculated earlier.

Outlet works consist of a reinforced-concrete box riser with a 15-in.-diameter reinforced-concrete pipe conduit that discharges water to Blind Brook. The main function of the outlet structure is to maintain normal pond water surface elevation (60.0 ft). During flooding, the outlet structure balances discharge so that the inflows (from Blind Brook and direct runoff) are detained, which allows for required storage within the pond.

An emergency spillway weir was provided to control discharge to and from the pond during extremely severe flood events. This weir should only function for storms of approximate return periods of 100 yr or greater.

Storage and net discharges from the pond during a storm become a function of adjacent area runoff and variable pond and Blind Brook water surface elevations. To route the floods, it was necessary to solve for pond water surface elevation using an iterative approach for successive time intervals. An algorithm was developed to determine storage-discharge values for a given set of conditions along Blind Brook, a given discharge hydrograph to the pond.

The basis for numerical solution to the routing problem is the following basic volume-balance relationship for inflow, outflow, and storage:

$$\bar{I}\Delta t - \Delta S = \bar{O}\Delta t \tag{2}$$

where

$\bar{I} = (I_1 + I_2)/2$ = average outflow rate,
$\bar{O} = (O_1 + O_2)/2$ = average outflow rate,
$\Delta S = S_2 - S_1$ = change in storage volume, and
$\Delta t = t_2 - t_1$ = routing period (T).

Subscripts 1 and 2 denote the beginning and end of the routing period. Restating and rearranging the basic equation yields the following routing formula:

$$(I_1 - O_1) + (2S_1/\Delta t) \mp (2S_2/\Delta t) + (O_2 - I_2) \tag{3}$$

Solution of this volume-balance equation requires that it be reduced to a function of a single dependent variable. In this case, Hp, the pond elevation at the end of each successive time interval, can be used to define inflow and outflow discharge rates as well as pond storage volumes. This was accomplished by making a simplifying pressure flow assumption with regard to flow at the 15-in. riser conduit and 24-in. inflow pipe and describing the proposed pond storage/elevation relationships using a linear approximation for storage-volume variation between specific elevation intervals. Inserting standard relationships into the equation yields the following working equations for the solution:

$$f(Hp) = \Big\{(2m_i Hp/\Delta t) + (2y_i Hp/\Delta t) \pm [Kr(Hp - Hr)^{0.5} \pm (Hp - Hw)^{0.5} - I_2 - A\Big\} \tag{4}$$

$$df(Hp)/dHp = (2m_i/\Delta t) + 0.5\, Kr\, (Hp - Hr)^{0.5} + 0.5 Kp\, (Hp - Hw)^{-0.5} \tag{5}$$

where

Hp = pond elevation;
Hw = Blind Brook headwater elevation at inflow pipe;
Hr = Blind Brook tailwater elevation at outflow riser conduit;
I_2 = direct runoff discharge to pond at end of time interval;
A = $I_1 - O_1 + (2S_1/\Delta t)$, a constant, calculated for start of specific time interval;
Kr = $C_d Ar(2g)^{0.5}$, a constant conveyance factor for calculating flow through the rise conduit;
Kp = $C_d Ap(2g)^{0.5}$, a constant conveyance factor for calculating flow through the inflow pipe;
Δt = time interval;
m_i = linear slope constant for storage-elelevation curve;
y_i = intercept constant for storage-elevation curve;
C_d = coefficients of discharge specific for each pipe, as a function of length, diameter, pipe material, and direction of flow
determined by using King and Brater Handbook of Hydraulics (8);
Ar, Ap = pipe areas; and
g = 32.2 ft/sec² gravity constant.

It should be noted that the first two terms of the right-hand side of Equation 4 establish storage volume. The third and fourth terms describer riser and inflow pipe hydraulics, respectively.

The signs of the second and third components of Equation 4 are shown as being variable because the relative elevation of the pond water surface (Hp), inflow pipe (Hw), and riser conduit (Hr) will change over the course of the design storm event. Although flow through the inflow pipe will be directed into the pond for a significant portion of the storm event, flow reversal will eventually occur as Blind Brook floodwater recedes below the elevation of the pond water surface. For certain portions of some events, flow reversal into the pond occurs at the riser location because Blind Brook flood elevations briefly exceed pond elevations there. For this reason, it was necessary to establish a basis for performing the routing under the full range of flow conditions that could occur during a particular event. The four different cases that are possible are presented in Table 8. Sign conventions are applied on the basis of the direction of discharge.

TABLE 8 Sign Conventions for Routing Solution

	Flow Conditions		Sign Convention	
Case	At 24-in. Inflow Pipe	At 15-in. Riser Conduit	Kp	Kr
1	Hw > Hp	Hr < Hp	–	+
2	Hw > Hp	Hr > Hp	–	–
3	Hw < Hp	Hr < Hp	+	+
4	Hw < Hp	Hr > Hp	+	–

Thus, under Cases 2 and 3, net discharge must be negative or positive, respectively, because Case 2 involves all flow going into the pond whereas Case 3 involves all flow going out of the pond. Under Cases 1 and 4, the sign of the net discharge is dependent on the relative magnitude of flow between the pond and Blind Brook at each of the structures.

For numerical solution of the routing formula, the basic formula, used by Newton's method to generate the next approximation of the zero root is

$$Hp_{i+1} = Hp_i - f(Hp_i)/f'(Hp_i) \qquad (6)$$

Successful convergence is accomplished when the absolute value of the numerical difference between successive approximations of Hp is smaller than a specified value, E. For this design analysis, convergence occurred when E was set at 10^{-4}.

Stage versus time relationships for the outlet and inlet structure points along Blind Brook were prepared for each design storm. These values were determined using the HEC-2 computer program for various flow discharges. Stage-discharge curves generated for each structure location were then correlated with runoff hydrographs for Blind Brook (7).

Stage-discharge curves used in determining Blind Brook elevations are shown in Figure 6. Hydrographs

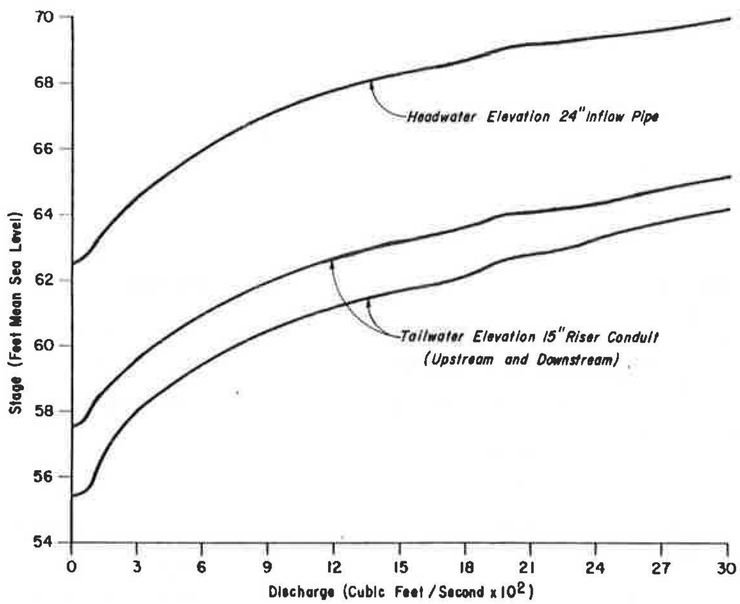


FIGURE 6 Blind Brook stage and discharge curves.

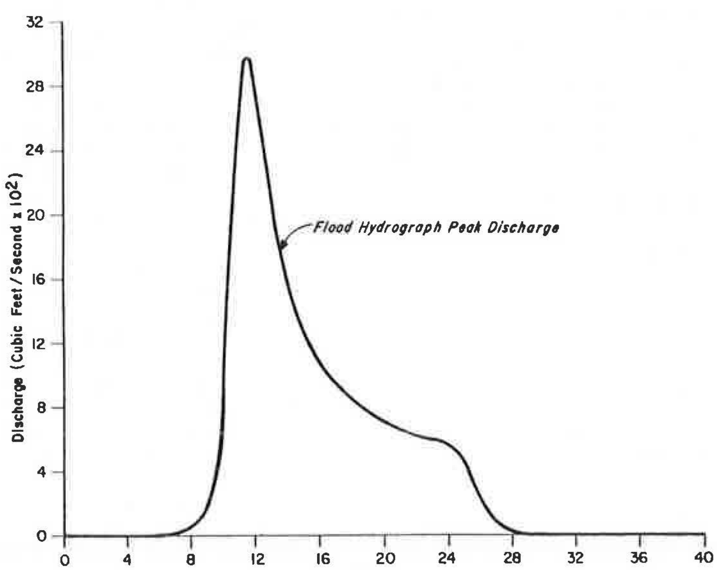


FIGURE 7 Blind Brook design hydrograph.

were used in developing Stage-time data used in flood routing, and the design hydrograph for the 100-yr storm in shown in Figure 7. The relationship of storage volume to water surface elevation for the pond, as used in the routing, is shown in Figure 8.

Flood routing data for the 100-yr storm event are given in Table 9, and are shown in Figure 9. Comparable flood routing data were also prepared for the 10-, 25-, and 50-yr storm events. Input for each time interval consisted of direct runoff discharged to the pond and the Blind Brook elevation at the inflow pipe and outlet conduit. Net discharges are expressed as the net flow through the combined system. Negative values indicate that inflow to the pond from Blind Brook exceeds loss from the pond through discharge. Generally, this reflects the condition when flow through the 24-in. diameter inflow pipe is greater than inflow passing out of the 15-in. diameter riser conduit. The case (1, 2, or 3) refers to the flow conditions described previously.

DISCUSSION OF RESULTS

The data that resulted from the routing indicated that construction of the detention pond would result in a dramatic mitigative effect of peak runoff from the proposed site. Peak flows would be reduced 77, 75, 71, and 66 percent for 100-, 50-, 25-, and 10-yr storms, respectively. Moreover, these peaks would occur at a point in time during the storm well after the time of peak flood occurrence in Blind Brook. At the expected time of peak, there is net negative discharge. In this manner, peak flow from the site would be further attenuated and discharge peaks greatly reduced.

Storage requirements have been met or exceeded for all conditions. During filling of the pond by diversion of flow from Blind Brook, peak flows in the stream are reduced, which further improves conditions downstream, in addition to the improvements already affected by site peak discharge attenuation.

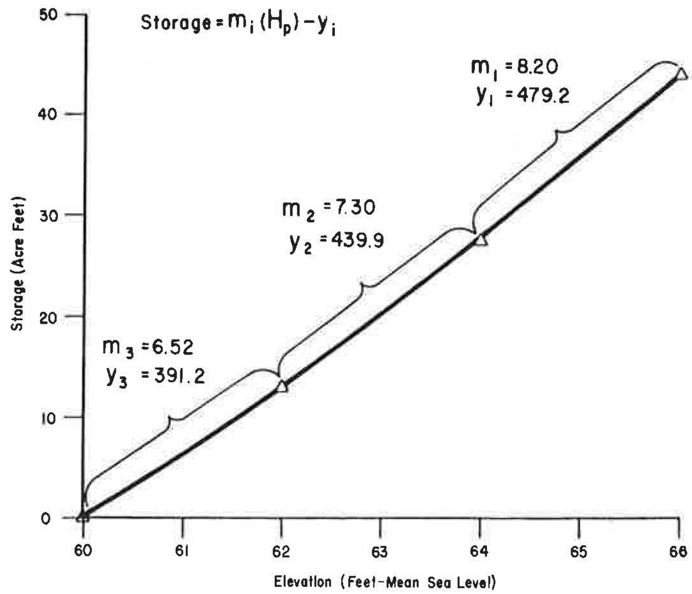


FIGURE 8 Detention pond storage versus elevation curve.

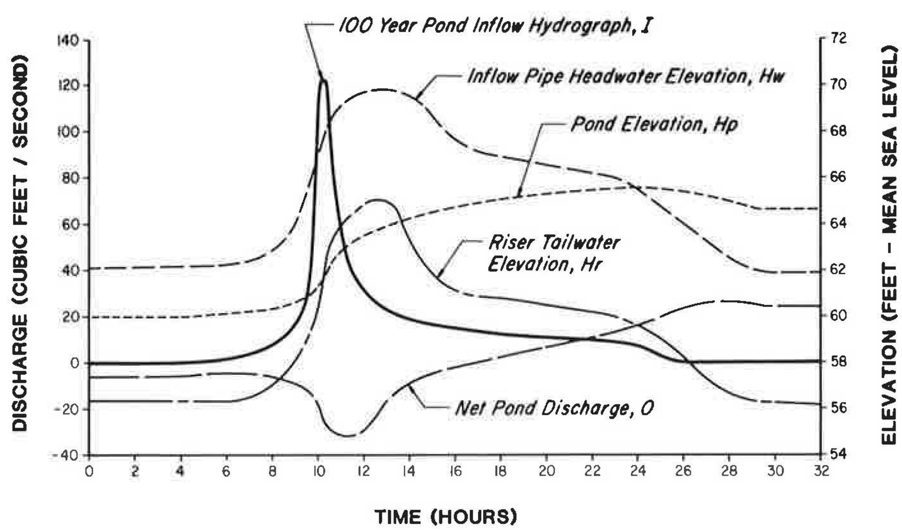


FIGURE 9 Detention pond flood routing—100-year storm.

TABLE 9 Detention Pond Flood Routing: 100-Yr Storm

Time (hr)	Runoff Discharge to Pond (ft^3/sec)	Headwater Elevation at Inflow Pipe (ft at mean sea level)	Tailwater Elevation at Riser Conduit (ft at mean sea level)	Case	Net Pond Discharge (ft^3/sec)	Pond Water Surface Elevation (ft at mean sea level)	Storage Volume (acre-ft)
4.3	1	62.1	56.5	1	-1.30	60.01	0.10
6.3	3	62.25	56.6	1	-1.53	60.10	0.65
8.3	12	62.9	57.3	1	-3.60	60.33	2.13
10.3	140	68.0	62.25	2	-26.43	61.86	12.13
11.3	46	69.0	64.2	2	-29.01	63.29	22.11
12.3	31	69.8	65.05	2	-28.09	64.01	27.66
14.3	21	68.2	62.35	1	-6.83	64.81	34.25
16.3	17	67.4	61.38	1	-0.83	65.27	38.01
18.3	14	66.8	60.90	1	3.44	65.55	40.31
20.3	12	66.5	60.55	1	6.28	65.72	41.67
22.3	11	66.1	60.25	1	8.88	65.69	41.46
24.3	8	65.4	59.70	3	18.77	65.55	40.32
26.3	0	64.0	57.75	3	26.82	65.14	36.94
28.3	0	62.2	56.5	3	31.71	64.53	31.93

The unique and innovative stormwater management system implemented for the General Foods site has effectively accomplished significant stormwater discharge and flood mitigation within all established policy and criteria. Since completion of this project in the spring of 1983, the detention pond is reported to have functioned well, in accordance with design parameters. General Foods has received correspondence from property owners downstream of their site, expressing appreciation for the observed mitigation of flooding along Blind Brook that resulted from the construction and operation of the detention pond.

Combining technology from established methods of stormwater management permitted development of a dynamic system to control increases in local stormwater runoff and provide additional flood control for a portion of the peak discharge on Blind Brook. In addition, the resultant detention pond is considered an aesthetically pleasing improvement to the site, which enhances the appearance of the General Foods corporate headquarters.

REFERENCES

1. Urban Hydrology for Small Watersheds. Technical Release 55. Soil Conservation Service, U.S. Department of Agriculture, Jan. 1975, 91 pp.
2. Tabular Method of Flood Routing. Technical Note--Engineering 20 (revision 2). Northeast Technical Service Center, Soil Conservation Service, U.S. Department of Agriculture, April 1978.
3. Standardized Rainfall Distribution and Dimensionless Unit Hydrographs in TR-20 and Structure Routing Computer Programs. Advisory ENG-FW-11. Northeast Technical Service Center, Soil Conservation Service, U.S. Department of Agriculture, Dec. 1977.
4. National Engineering Handbook. Section 4: Hydrology. Soil Conservation Service, U.S. Department of Agriculture, Aug. 1972.
5. HEC-2 Water Surface Profiles Computer Program 723-X6-L202A User's Manual. Hydrologic Engineering Center, Corps of Engineers, U.S. Department of the Army, Davis, Calif., Aug. 1979.
6. Flood Insurance Study--Town of Rye, New York. Federal Emergency Management Agency, Federal Insurance Administration, Sept. 1980, 32 pp.
7. Computer Program for Project Formulation Hydrology. Technical Release 20, Engineering Division, Soil Conservation Service, U.S. Department of Agriculture, Mar. 1969.
8. E.F. Brater and H.W. King. Handbook of Hydraulics. McGraw-Hill Book Co., New York, 1976, pp. 4-24.

Forecasting Pollutant Loads From Highway Runoff

KENNETH D. KERRI, JAMES A. RACIN, and RICHARD B. HOWELL

ABSTRACT

Forecasting regression equations for estimating pollutant loads in runoff from highways are developed in this paper. Data were collected during the runoff seasons at completely paved urban highway sites in Redondo Beach, Walnut Creek, and Sacramento, California. Information was also obtained from a rural site near Placerville. Rainfall and runoff were monitored continuously. Bubbler flow meters were used with automatic sequential samplers so that storm water samples could be collected to characterize entire storm events. The constituents that were analyzed were boron, total lead, total zinc, nitrate (nitrogen), ammonia (nitrogen), total Kjeldahl nitrogen, total phosphorus, dissolved orthophosphate, oil and grease, nonfilterable residue, filterable residue, total cadmium, and chemical oxygen demand. The number of vehicles during the storm was evaluated and accepted as a satisfactory independent variable for estimating the loads of total lead, total zinc, filterable residue, chemical oxygen demand, and total Kjeldahl nitrogen. The total residue was evaluated and accepted as a satisfactory independent variable for estimating total zinc, nonfilterable residue, and chemical oxygen demand. Estimates by using these equations should be limited to highways with average daily traffic of at least 30,000 vehicles. The numbers of antecedent dry days was found not to be a satisfactory independent variable.

A method is needed to forecast expected pollutant loads in the runoff from highway surfaces. When an existing highway is upgraded or a new highway is constructed, natural runoff patterns are altered. These alterations can cause significant changes in flow rates and runoff water quality.

Runoff from highway surfaces carries potentially harmful pollutant loads to nearby surface waters. These pollutants may be deposited on the highway surfaces from vehicles traveling on the highway, from winds, and from fallout of air pollutants.

The pollutants may be dissolved in the runoff water or carried as particulate matter. An adverse impact on the receiving water may result from the toxic (heavy metals), oxygen-consuming, biostimulation (nutrients), or aesthetic (oil and grease) characteristics of the pollutant. The magnitude of the impact may be a function of the concentration of the pollutant or the total quantity (load) of the pollutant that reaches the receiving waters during a storm event or that is accumulated over a period of years.

The objectives of this research project are (a) identify those pollutants in highway runoff that may cause an adverse impact on receiving waters, and (b) develop forecasting regression equations that can be used in predicting the expected pollutant loads in highway runoff. To attain these objectives, one rural site and three urban sites were selected. The rural site was located near Placerville and the urban sites were located near Redondo Beach, Walnut Creek, and Sacramento (Figures 1 and 2).

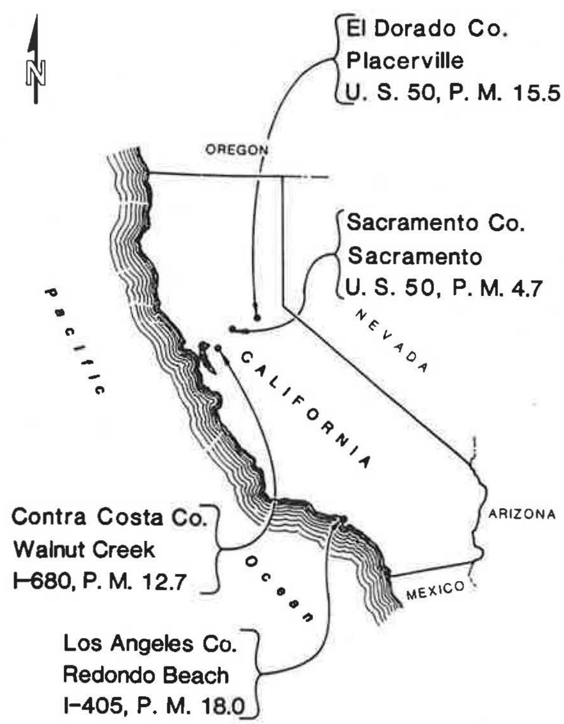
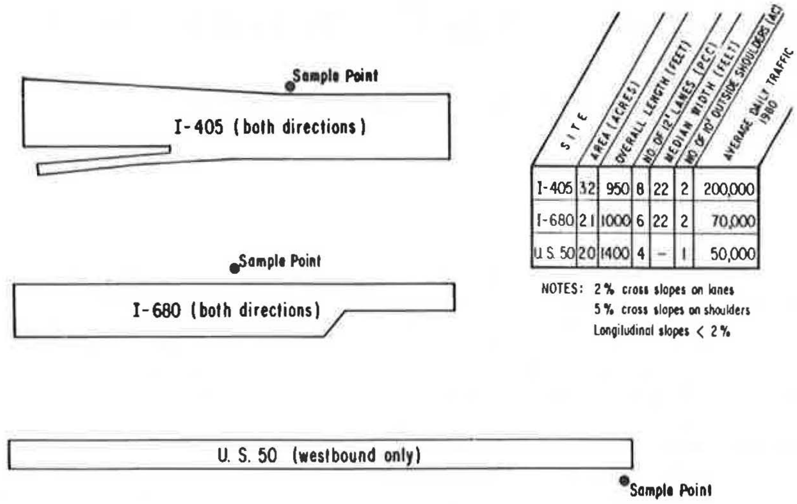


FIGURE 1 Runoff sites.

RESEARCH PROCEDURES

Because of the complexity of interactions among pollutants, rainfall, runoff, highway design, operating vehicles, surrounding land use, and maintenance practices, regression analysis was the technique chosen for building the forecasting equations. The main thrust of the regression analysis was to find a suitable independent variable that could be used to quantify the response variables. The response variables were the selected pollutants that had already been identified in past research by the California Department of Transportation (Caltrans) and others, which can be generally classified as heavy metals (toxicants), oil and grease (aesthetic), nutrients (biostimulants), and residue (particulate material).



SITE	AREA (ACRES)	OVERALL LENGTH (FEET)	NO. OF 12' LANES (PCC)	MEDIAN WIDTH (FEET)	NO. OF 10' OUTSIDE SHOULDERS (AC)	AVERAGE DAILY TRAFFIC 1980
I-405	32	950	8	22	2	200,000
I-680	21	1000	6	22	2	70,000
U.S. 50	20	1400	4	-	1	50,000

FIGURE 2 Simplified plan views and runoff sites.

Data Sources

Three sets of data were used for developing the forecasting regression equations. The first was collected by Caltrans from 1975 through 1978 (1). The data consist mainly of instantaneous flow observations and water quality measurements of manually obtained samples. These data were screened to select constituents for additional sampling in the current study. They were also used for preliminary investigations. The runoff sites were at I-405 in Redondo Beach, I-680 in Walnut Creek, and US-50 in Placerville (Figure 1). The second set of data was collected by Caltrans specifically for this study at I-405 in Redondo Beach and I-680 in Walnut Creek during the 1980-1981 wet season. The data consist mainly of continuous flow observations and discrete water quality measurements of composited samples of entire storms. These data were used to build the tentative regression equations.

The third set of data was collected by Caltrans, but was analyzed and reported by Envirex, Inc. (2-4) in cooperation with FHWA. The runoff site was at US-50 in Sacramento. Data were collected from 1979 through 1981. The data consist mainly of continuous flow observations and discrete water quality measurements of composited samples of entire storms. These data were used to evaluate the tentative regression equations based on Redondo Beach and Walnut Creek data.

Flows were measured and recorded continuously at all sites using Parshall flumes. Water quality samples were collected using automatic sequential discrete samplers. An electronic probe sensed the start of runoff from each storm and transmitted a signal that activated the samplers. The samplers were programmed to collect samples either at 15-min intervals or after every 100- or 300-ft^3 of flow through the Parshall flume (5). Similar results were obtained by using either the time or volume incremental method of sampling.

Selection of Constituents to Sample: Dependent Variable

Seven storms were evaluated to determine whether significant concentrations of any of 31 constituents existed in the storm water before it entered the receiving waters, where further dilution would reduce concentration levels or the addition of a pollutant could increase concentration levels. The significance criteria were based on the maximum observed concentration, which had to be within 50 percent of critical concentration reported by the U.S. Environmental Protection Agency (6) or the California Water Resources Control Board (7).

Lead (Pb), zinc (Zn), cadmium (Cd), and oil and grease were chosen for study because they are vehicle related. Total residue (TR) was selected because it is associated both with vehicles and with local air particulate deposition. Nitrate-nitrogen, ammonia-nitrogen, total Kjeldahl nitrogen (TKN), total phosphorus, and orthophosphate were studied because these constituents may collect on traveled surfaces and shoulders. Boron was studied because of its potential impact on vegetation. Cadmium was selected for the 1980-1981 sampling program; however, cadmium testing at the Walnut Creek site was discontinued after the first major storm on December 10, 1980, because values dropped below detection limits after the initial runoff. Analysis of the 1975-1978 data revealed a "first flush" pattern: sulfate, iron, chromium, copper, manganese, and nickel concentrations were not within 50 percent of the critical concentrations listed in Table 1.

DEVELOPMENT OF POLLUTANT LOAD FORECASTING EQUATIONS

Analysis of the 1975-1978 data (1) revealed that insufficient information was collected during storm events. To determine pollutant loads in either pounds or kilograms of each specific pollutant per storm event, the storm hydrograph and the pollutant concentrations (pollutograph, a plot of pollutant concentrations versus time) during the runoff period must be known. Continuous flow recorders and automatic samplers were used to collect these data. Samples for oil and grease were collected manually by field personnel during storm events to obtain representative samples.

Number of Vehicles Before the Storm as an Independent Variable

Hypothesis testing of regression equations by using number of vehicles before the storm as an independent variable and the constituent load as a dependent variable showed no statistical significance.

TABLE 1 Constituents to be Considered for Further Study

Constituent	Criterion	Critical Value
Boron[a]	Crops	750 μg/liter
Sulfate	Water supply	250 mg/liter
Iron	Water supply	300 μg/liter
Lead[a]	Water supply	50 μg/liter
Zinc[a]	*Salmo gairdneri*	10 μg/liter
Nitrate-nitrogen[a]	Aquatic growth	300 μg/liter
Total Kjeldahl nitrogen[a]	Aquatic growth	600 μg/liter
Ammonia[a]	Aquatic life	20 μg/liter
Total phosphorus[a]	Aquatic growth	10 μg/liter
Dissolved orthophosphate[a]	Aquatic growth	10 μg/liter
Oil and grease[a]	Water supply	Virtually free from oil and grease
Total residue[a]	Water supply	250 mg/liter for Cl
Total nonfilterable residue[a]	Water supply	Variable
Chemical oxygen demand[a]	Treatment plant effluent	50 mg/liter
Conductivity[a]	Aquatic life	1,000 μmhos
pH[a]	Special treatment required	
Cadmium[a]	Salmonid	0.4 μg/liter
Chromium	Water supply	50 μg/liter
Copper	*Salmo gairdneri*	2.0 mg/liter
Manganese	Water supply	50 μg/liter
Nickel	Water supply	100 μg/liter

[a]Constituent was selected for 1980-1981 sampling program because the observed concentrations were within 50 percent of the critical value shown.

The results of sweeping/flushing studies performed at the US-50 site in Sacramento by Envirex Inc. (2-4) showed that the active freeway lanes do not retain significant amounts of pollutants. Furthermore, the Envirex dustfall transect study in Sacramento indicated that particulates were blown off the traveled lanes to the shoulders and beyond. Evidently, during the antecedent dry period, the freeways are continuously swept by the traffic-generated turbulence; thus, it was not surprising that the correlation of pollutant loads with amount of traffic before the storm [average daily traffic (ADT) times dry days] showed no statistical significance. During dry periods, there apparently is a greater adherence of materials to the engine, undercarriage, and wheel walls of vehicles, whereas during a storm or wet periods, there is more splashing and washing of these materials from the vehicles.

Number of Vehicles During the Storm as an Independent Variable

A study of Lead Emissions and Washoff (5,8) showed that a significant fraction of lead emitted from vehicles during a storm correlated well with lead in runoff. Further studies using the number of vehicles during the storm (VDS), as the independent variable were made by using Equation 1. Equation 1 is the general form of the line:

$$CL = a + b(VDS) \quad (1)$$

where CL is the cumulative constituent load, and a and b are the regression coefficients. Vehicles were counted on an hourly basis in this research project to match, as closely as possible, the times from start to end of runoff. Vehicle counts were obtained from the Traffic Operations Branches of the respective Caltrans districts in which the sampling was performed.

The ideal forecasting equation is a regression equation in which the independent variable is easily sampled and quickly and inexpensively measured. When a highway project requires that nonpoint source pollution via runoff be addressed, the relatively inexpensive tests of TR, nonfilterable residue (NR), and filterable residue (FR) could be performed and estimates of the other constituents could be obtained from the prediction equations. Subsequent evaluation studies led to the formulation of Equation 2 in which no transformations (normalizing) were applied to the data.

$$CL = a + b(TR) \quad (2)$$

where CL is cumulative constituent load, and a and b are regression coefficients (a represents an initial load in grams, and b is the fraction of constituent washed off the highway during a storm).

RESULTS

Linear regression equations were tested and evaluated by using number of vehicles during the storm as an independent variable to quantify the loads of the following constituents: Pb, Zn, FR, chemical oxygen demand (COD), and TKN. In addition, linear regression models were tested and evaluated by using the total residue as an independent variable to quantify the loads of the following constituents: Zn, NR, and COD.

The equations for each constituent are shown in Figures 3 through 10. Each figure shows a plot of the tentative equation (Line A), which was based on observations from Redondo Beach and Walnut Creek, in addition to the pooled equation (Line B), which in-

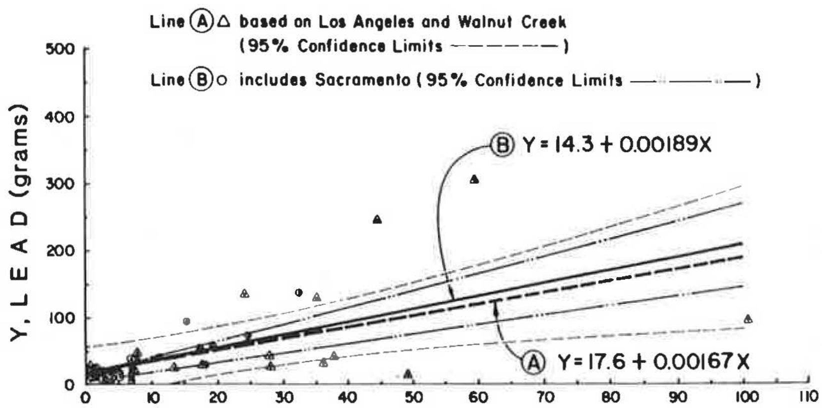


FIGURE 3 Lead versus vehicles during storm.

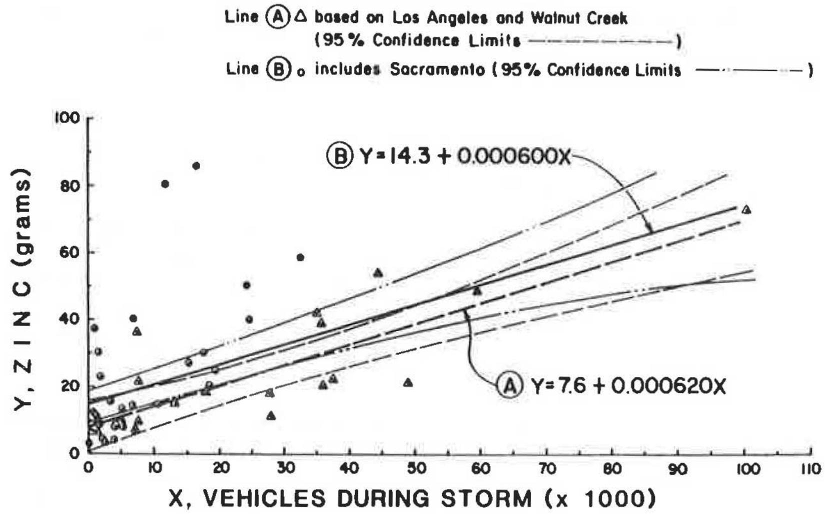


FIGURE 4 Zinc versus vehicles during storm.

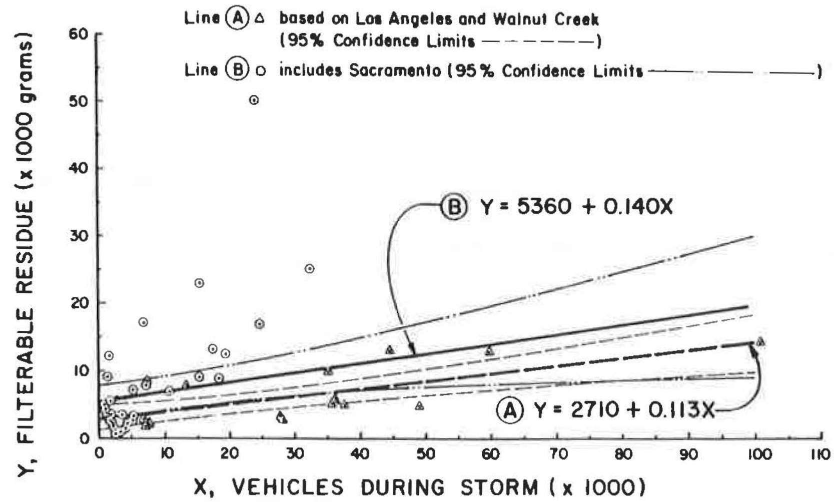


FIGURE 5 Filterable residue versus vehicles during storm.

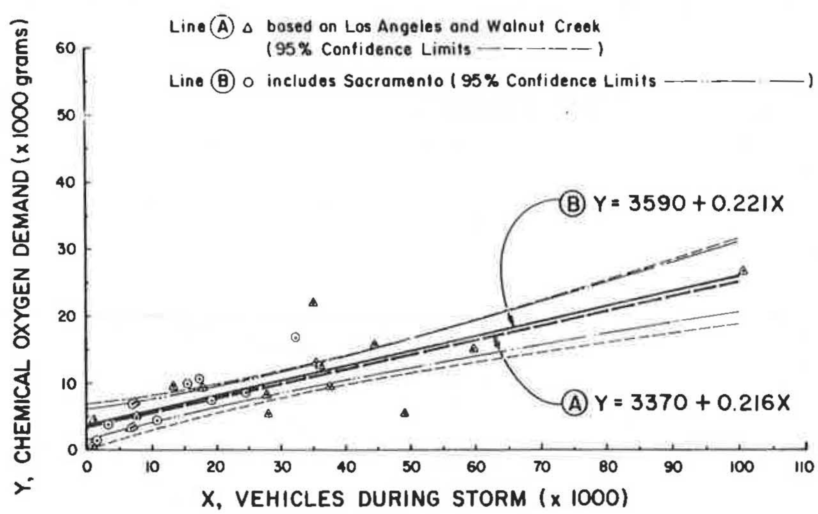


FIGURE 6 Chemical oxygen demand versus vehicles during storm.

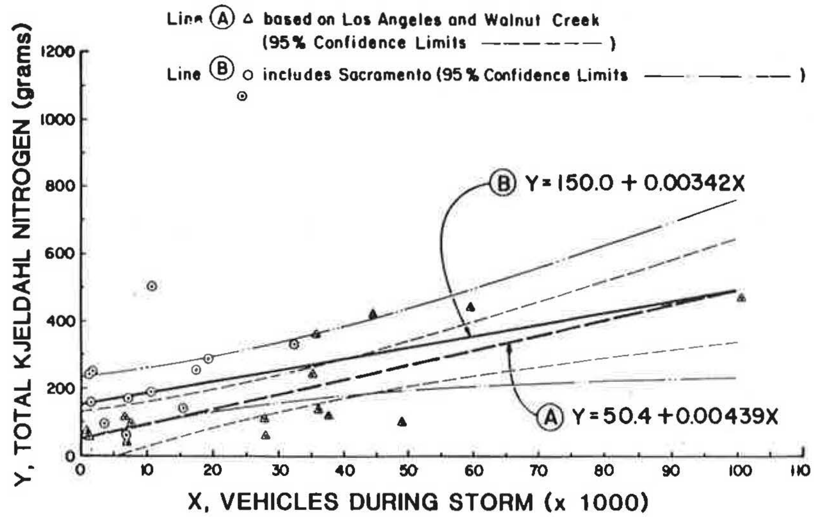


FIGURE 7 Total Kjeldahl nitrogen versus vehicles during storm.

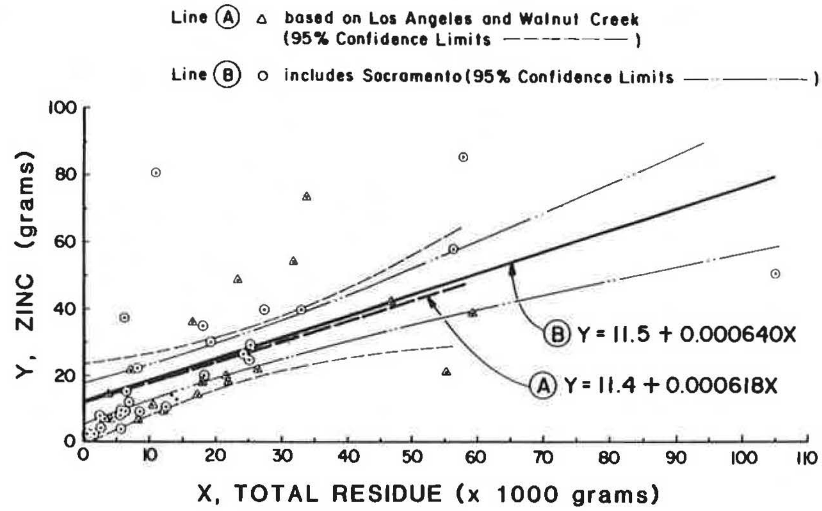


FIGURE 8 Zinc versus total residue.

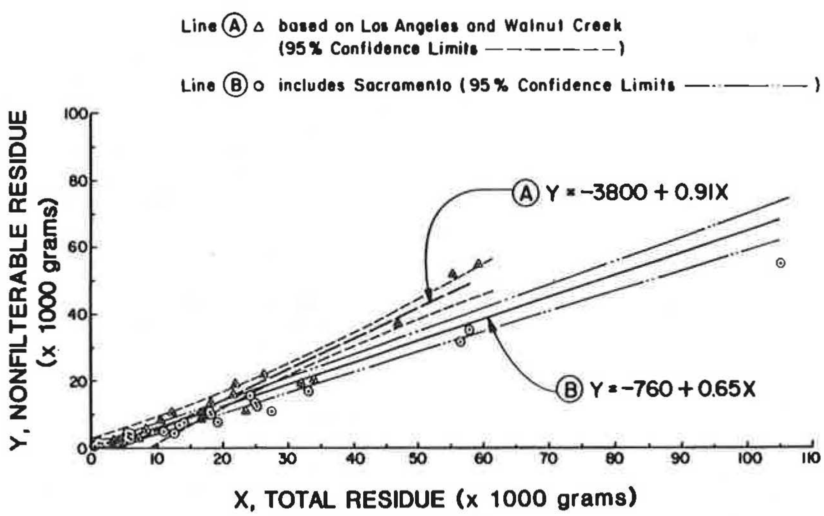


FIGURE 9 Nonfilterable residue versus total residue.

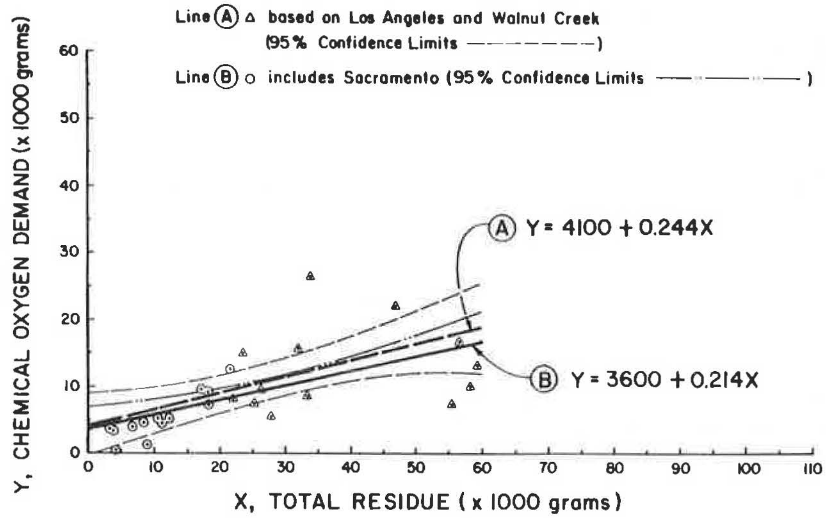


FIGURE 10 Chemical oxygen demand versus total residue.

cluded observations from Sacramento. The 95 percent confidence limits are plotted for each line. The equations are statistical representations that were based on continuous observations for each storm event of constituents found in runoff from urban highways in California.

The equations may be applied for 100-percent-paved highways that have the same general site characteristics as the sites from which the observations were obtained. Figure 2 shows a simplified plan view of each of the completely paved test sites. Longitudinal slopes were generally less than 2 percent, so that the times of travel of runoff, which originated at the farthest point on the drainage catchment, to the sampling location were less than 30 min.

The constituents for which no linear relationships were found by using Equation 1 are boron, cadmium, nitrate-nitrogen, ammonia-nitrogen, total phosphorus, dissolved orthophosphate, NR, TR, and oil and grease. In addition, no correlations were found by using Equation 2 for the following constituents: boron, cadmium, lead, nitrate-nitrogen, TKN, ammonia-nitrogen, total phosphorus, dissolved orthophosphate, FR, and oil and grease.

Summary of Evaluations

Use of the number of vehicles during the storm as a variable (Equation 1) was found to be acceptable by t-testing the equality of slopes of the equations for total Pb, total Zn, FR, COD, and TKN. Total residue (Equation 2) was found to be acceptable by t-testing the equality of the slopes of the equations for total Zn, NR, and COD.

Analysis of laboratory test results from the runoff at each of the four sites (1-5,9,10) revealed that none of the pollutants that were studied produced levels of contaminants that exceeded the water quality criteria in Table 1 or that created detrimental impacts when they eventually reached nearby receiving waters.

EXPLANATION AND PROCEDURE FOR USING THE FORECASTING EQUATIONS

Before the regression equations are used to compute constituent loadings, there are three criteria to examine: (a) there must be a sensitive receptor nearby (e.g., a stream that supports aquatic life or is a municipal water supply); (b) the ADT must exceed 30,000 vehicles; and (c) the average annual rainfall in the area should be between 18 and 24 in.

Because the drainage details are not known in the advance stages of a highway project, the following assumptions and procedures are used to forecast constituent loads and flow-weighted concentrations:

1. The future vehicle fleet and fuels used are approximately the same as in the years of actual data collection (1979-1981).
2. The highway is in an urban setting in an arid or semiarid region of the United States.
3. The median, traveled lanes, and shoulders are 100 percent paved.
4. The assumed drainage area is the actual proposed width of pavement times an assumed length. The drainage area should be between 2 and 4 acres to correspond to the drainage areas used for the research sites. The actual site configurations and characteristics are as follows (see Figure 2 also):

Physical Characteristics	Los Angeles	Walnut Creek	Sacramento
Area (acres)	3.2	2.1	2.0
Lane miles	1.4	1.0	1.1
Gutter miles	0.70	0.56	0.27

5. Runoff from the assumed drainage area is conveyed via open channels to a single point of discharge. (Runoff quantity and quality from the unpaved area adjacent to the paved area was excluded from this study.)
6. A runoff coefficient of 0.90 is used to compute the cumulative runoff volume because the drainage area is completely paved.
7. The hydrologic records are analyzed to determine (a) the expected number of storms per year with sufficient duration and intensity occurring during both the a.m. and p.m. peak traffic to wash off the gutter load and contributions to pollutant runoff from traffic traveling through the site and (b) the expected precipitation per storm in (a). Precipitation data can be obtained from hydrologic data available from the U.S. Geological Survey, National Oceanographic and Atmospheric Administration, or state water resource agencies (11). Rainfall intensity may be a key factor at some sites; however, this study considered the volume of runoff that results from both the intensity and duration of a

storm. Various situations will require appropriate analysis and the application of hydrologic data.

8. Because the storm duration includes both the a.m. and p.m. peak traffic, the projected ADT is used to compute constituent loads by using the following linear regression equations, which were evaluated and found to be acceptable:

$$Pb = 14.3 + 0.00189(ADT) \quad (3)$$

$$Zn = 14.3 + 0.00060(ADT) \quad (4)$$

$$FR = 5360 + 0.140(ADT) \quad (5)$$

$$COD = 3590 + 0.221(ADT) \quad (6)$$

$$TKN = 150 + 0.00342(ADT) \quad (7)$$

where Pb, Zn, FR, COD, and TKN are the cumulative loads in grams per storm. The intercepts represent initial dry loads in grams, and the slopes represent the washoff rate of constituent in grams per ADT during a storm. The intercepts for FR and COD indicate that a first flush of particulate matter can be expected to consist of some organic materials.

9. To forecast an annual load, each of the daily loads (item 8) is multiplied by the expected number of 1-day events per year (item 7a) to arrive at an annual load.

10. The flow-weighted concentration is computed by dividing the daily event load in item 8 by the 1-day cumulative runoff volume (use item 7b).

11. The following linear regression equations use total residue to calculate constituent loads in grams per storm:

$$Zn = 11.5 + 0.00064(TR) \quad (8)$$

$$COD = 3600 + 0.214(TR) \quad (9)$$

$$NR = -760 + 0.65(TR) \quad (10)$$

The intercepts represent the initial dry loads in grams, and the slopes represent the fraction of constituent found in the TR that is washed from the pavement during the storm. Because TR is an independent variable for which no easy future value can be obtained, the following procedure is suggested. Substitute the values of total Zn computed from Equation 4 and COD computed from Equation 6 (which are based on ADT during storm) in Equations 8 and 9. Solve these two equations for the independent variable, TR. Use the average of the two calculated values of TR to compute the NR load using Equation 10. Then, compute the flow-weighted concentration as in item 10.

12. The final step of the procedure is to check the computed loads and flow-weighted concentrations. The check is to ensure that the computed values are bounded by the field observations. Table 2 shows the limits of the observed concentrations and loads.

TABLE 2 Limits of Observed Concentrations and Loads of Single Events

	Concentration (mg/liter)		Load (gr)	
Constituent	Low	High	Low	High
Total lead	0.17	4.10	4.0	304
Total zinc	0.10	1.80	2.0	84
Filterable residue	16	461	428	50,167
Chemical oxygen demand	23	724	382	26,344
Total Kjeldahl nitrogen	0.1	14.0	34	1,070
Nonfilterable residue	18	2,660	143	55,259

Final water quality assessment must be made by applying values of the pollutant loads and concentrations to the receiving waters and by determining the resulting impacts. To assess the effects of the constituent load on nearby receiving waters, the load must be routed through the drainage system and, ultimately, to the receiving water. Along the way, runoff from other sources may be encountered. To conduct an environmental assessment, these other sources must be included along with dilution factors for the highway runoff in terms of the receiving waters.

Inclusion of mitigation measures in transportation projects to reduce the influence of pollutants from paved highway surfaces should be based on findings from analyses that are performed in accordance with the procedures described in this paper. The following list should be considered in determining mitigation measures:

1. A potential mitigation measure is to route direct runoff through grass-covered drainage courses to remove particulate matter, heavy metals, trace organics, toxicants, and oxygen-consuming materials (12).
2. Where mitigation measures are needed, proper designs should be based on pollutant loading analyses to provide a cost-effective measure to protect the aquatic receptor.
3. Inclusion of mitigation measures that do not improve water quality on projects should be avoided to reduce unnecessary costs.

CONCLUSIONS

The following conclusions were reached from this research project:

1. Urban highways in California that are operated under normal conditions (i.e., no accidents or chemical spills) do not produce large amounts of pollutant constituents during storm runoff events. The findings of the research indicate that for highway segments that drain between 2 and 4 acres of completely paved areas, and have six to eight traveled lanes, the constituent pollutant loads in runoff water are sufficiently low so that costly treatment facilities are not needed to meet water quality objectives.

2. Equations to estimate the cumulative loads of the following pollutants were found to be statistically significant at the 5 percent level on a storm event basis when correlated with the number of vehicles during the storm (pollutants included COD, FR-dissolved solids, total Pb, TKN, and total Zn) and with total residue (pollutants included COD, NR-suspended solids, and total Zn).

The number of dry days between storm events and the corresponding cumulative traffic volume before the storm were found to be not statistically significant for quantifying cumulative constituent loads. Apparently, traffic-generated turbulence tends to continuously sweep the traveled lanes and shoulders that were studied in this project.

After the initial pavement and gutter loads are washed off, vehicles traveling on the highway will continue to release pollutant constituents. Pollutants will also be reaching the highway surface from atmospheric fallout and surrounding land-use activities. Because constituents are continuously being added to the runoff, the use of an exponential washoff equation is not adequate. Instead, a linear approximation is appropriate.

3. No statistically significant correlations at the 5 percent level of significance were found with any of the independent variables examined for the

following constituent loads: boron, cadmium, nitrate-nitrogen, ammonia-nitrogen, total phosphorus, dissolved orthophosphate, oil and grease.

The following constituents exhibited a first flush pattern with relatively insignificant loads and concentrations: sulfate, iron, chromium, copper, manganese, nickel, bicarbonate ion, carbonate ion, calcium, magnesium, chloride, mercury, molybdenum, potassium, silica, and sodium.

RECOMMENDATIONS

The following recommendations are made:

1. Determination of constituent loads for COD, FR, total Pb, TKN, and total Zn from pavement highway surfaces should be made for proposed highway projects where the ADT is at least 30,000 vehicles and a nearby sensitive environmental receptor, such as a stream, river, or lake, exists. These determinations can be made by using Equations 3-10.
2. The regression equations developed in this research can be used for calculating constituent loads from the paved traveled way and shoulder area. To assess the effects of the constituent load on nearby receiving waters, the load must be routed through the drainage system and, ultimately, to the receiving water.
3. The constituent regression coefficients of the equations should be reevaluated in the future as alternative fuel sources and transportation designs, modes, and operations change. Besides quantifying constituent loads, a future monitoring study of transportation runoff waters should include monitoring of vegetation and aquatic life so that mitigation measures can be designed that are compatible with transportation facilities. Also, future studies should assess prior research efforts in this area and be aware of the pragmatic limitations of the research.

ACKNOWLEDGMENTS

The research presented in this paper was accomplished under an FHWA Highway Planning and Research Program project at Caltrans (5).

REFERENCES

1. R.B. Howell. Water Pollution Aspects of Particles Which Collect on Highway Surfaces. Report FHWA/CA/TL-78-22. Transportation Laboratory, California Department of Transportation, Sacramento, July 1978, 149 pp.
2. Sources and Migration of Highway Runoff Pollutants. Monthly Progress Report 32. Envirex Inc., Milwaukee, Wis., June 1980, 14 pp.
3. Sources and Migration of Highway Runoff Pollutants. Monthly Progress Report 36. Envirex Inc., Milwaukee, Wis., Oct. 1981, 27 pp.
4. Sources and Migration of Highway Runoff Pollutants. Monthly Progress Report 39. Envirex Inc., Milwaukee, Wis., Jan. 1981, 33 pp.
5. J.A. Racin, R.B. Howell, G.R. Winters, and E.C. Shirley. Estimating Highway Runoff Quality. Report FHWA/CA/TL-82-11. Transportation Library, California Department of Transportation, Sacramento, 1982, 105 pp.
6. Quality Criteria for Water. Office of Water Planning and Standards. U.S. Environmental Protection Agency, 1976, 256 pp.
7. J.E. McKee and H.W. Wolf. Water Quality Criteria. California State Water Resources Control Board, Sacramento, 1963, 548 pp.
8. User Manual, LEAD1, Motor Vehicle Lead Emission Rates. Transportation Library, California Department of Transportation, Sacramento, July 1979, 32 pp.
9. Envirex Inc. (A Rexnord Company). Constituents of Highway Runoff (6 vols.). Reports FHWA/RD-81/042 through 047. Envirex Inc., Milwaukee, Wis., 1981, 714 pp.
10. G.R. Winters and J.L. Gidley. Effects of Roadway Runoff on Algae. Report FHWA/CA/TL-80/24. Transportation Laboratory, California Department of Transportation, Sacramento, June 1980, 241 pp.
11. J.D. Goodridge. Rainfall Analysis for Drainage Design, Vol. II, Long Duration Precipitation Frequency Data. Bull. 195. California Department of Water Resources, Sacramento, Oct. 1976, 381 pp.
12. B.L. Chaplin and F.E. Conroy. Plan for Implementation of Highway Runoff Water Quality Research Findings. Report WA-RD-39.17. Washington Department of Transportation, Olympia, Wash., Jan. 1984, p. 4.

Assessing the Impacts of Operating Highways on Aquatic Ecosystems

RICHARD R. HORNER and BRIAN W. MAR

ABSTRACT

A protocol has been developed for assessing the impacts of highway operations and maintenance and determining the need for impact mitigation measures. The general strategy applies nationally, and specific elements of the method have been developed for the state of Washington and other Pacific Northwest locations on the basis of comprehensive research that was conducted in that region on highway runoff water quality. The basic premise of the protocol is that the highway impact on the receiving water can be assessed most realistically in the context of the aggregate burden that is created by all activities in the watershed. By using an initial screening process a determination can be made as to whether or not a case is likely to have an insignificant impact. Substantial resources are expended on assessing only those cases that may have a significant impact on aquatic ecosystems. Those cases are subjected to analyses of both cumulative pollutant loadings and changes in pollutant concentrations in the receiving waters, which emphasize the most critical conditions under the circumstances. Mitigation is considered in both steps. The Washington results were employed to develop a deterministic model for the pollutant loading analysis and a probabilistic procedure for the pollutant concentration assessment. The protocol offers opportunities to forecast potential aquatic impacts of a highway at an early stage of project development and to allocate impact mitigation measures on the basis of need. This advance improves the cost-effectiveness of stormwater runoff management and aids in avoiding the expense and delay of legal challenges to highway agency actions that have potential water quality impacts.

The National Environmental Policy Act (NEPA) of 1969 requires that an environmental assessment be made of the anticipated consequences of each significant federal action. Beyond this and a few other general provisions, however, the law and regulations that are promulgated under its authority have provided little distinct guidance on impact assessment methodology. Various agencies have, however, developed guidelines for preparing environmental impact statements (EISs) for projects under their regulatory jurisdictions. In both the general situation and the highway case in particular (1), these guidelines usually concern the content of EISs and leave the selection of assessment procedures to the analyst.

A substantial amount of the applied environmental research performed since the adoption of the NEPA has had as an implicit objective the enhancement of abilities to conduct environmental impact assessments. Although many useful methods have resulted from these efforts, there have been few attempts to apply the knowledge gained to developing comprehensive assessment protocols. Even rarer has been the implementation of these research results in the practices of organizations that must prepare or evaluate EISs.

A comprehensive protocol has been developed for assessing impacts on surface waters that receive storm runoff from operating highways. The general philosophy of this protocol applies nationally, and specific elements have been developed for the state of Washington on the basis of a large amount of highway runoff water quality research conducted there. A handbook was prepared to provide step-by-step guidance for impact analysis (2). With the assistance of the researchers, the Washington State Department of Transportation is in the process of implementing the protocol in its practices (3). Following a review of the available techniques for assessing aquatic impacts of highway operations, this paper contains a discussion on the generalized protocol and its rationale as well as illustrations of its application with the specific procedures developed from the Washington results.

TECHNIQUES FOR ASSESSING AQUATIC IMPACTS OF OPERATING HIGHWAYS

Highway operations potentially affect receiving waters through peak flow increases, degradation of water quality, and modification of biotic habitats. Various hydrologic models are available to estimate highway runoff flow rates for design storm conditions and the resulting effects on stream discharges. A number of possible stormwater runoff contamination sources exist, including vehicular and atmospheric deposition, pavement wear, and various maintenance operations. Most techniques that can be applied to impact assessment reflect the overall quantities of pollutants that may be present, despite the fact that contributions from the respective sources vary spatially and temporally, both seasonally and annually.

Highway runoff also potentially affects receiving water quality over both the short and the long term. Short-term effects would be a function of high pollutant concentrations (pollutant mass per unit water volume) during individual runoff events (e.g., acute toxicity to aquatic biota). Long-term effects would be created by cumulative pollutant loadings (pollutant mass per unit time). Examples are sediment accumulation and seasonal nutrient loading to a lake. Most of the available aquatic impact assessment tools represent long-term loadings. However, most of the knowledge of aquatic ecosystem response and most water quality criteria issued as regulations are of a short-term nature.

The development of techniques that can be used to assess water quality impacts of operating highways has a very brief history. Sartor and Boyd (4) and Pitt and Amy (5) published data on pollutant accumulations on urban streets. Neither group attempted to determine their transport in storm runoff, however. The first efforts to characterize the runoff from operating highways were by Sylvester and DeWalle (6) and Soderlund and Lehtinen (7), who derived mass loadings per unit highway surface area. The Municipality of Metropolitan Seattle (8,9) introduced traffic as a variable, expressing freeway runoff pollutant loading data normalized on the basis of vehicle counts. Shaheen (10) also found traffic to be a key variable and derived linear regression equations to estimate pollutant loadings on the basis of traffic counts, vehicle deposition rates, and background pollutant levels. Envirex Inc. conducted extensive highway runoff studies at five sites east of the Rocky Mountains for FHWA. That work concluded with the development of a deposition model to predict the accumulation of pollutants in the periods preceding storms and a washoff model to forecast contaminant removal in the runoff, both on a total mass basis (11). These models were formulated for individual storm events.

Although all of these efforts yielded techniques amenable to aquatic impact assessment, none were formulated in a specific protocol for this purpose. This research effort had as a major objective the development of such a protocol and the methods necessary to apply it to problems in Washington State. The research effort was comprehensive, involving investigation of highway runoff pollutant sources, transport, fate, effects, and control. Many of the findings have been presented elsewhere (12-20), and the results of the most direct applications to impact assessment will be highlighted in this paper.

Key developments in this research were a cumulative pollutant loading model and a probabilistic approach for assessing short-term effects. The loading model consists of two components: (a) a simple algebraic equation that establishes cumulative total suspended solids (TSS) loading as a result of routine highway operation as a function of vehicles traveling during storm periods, and of runoff coefficient (ratio of runoff volume: precipitation volume), and (b) a series of multipliers for estimating loadings of other pollutants from TSS. Vehicles during storms are apparently important in controlling pollutant loading because of the spray washing that loosens contaminants deposited on vehicle undersides during dry weather. The California Department of Transportation (21) also found that introduction of this variable produced statistically significant results in its data analysis. Other pollutant loadings can be predicted from TSS because the majority of these pollutants are associated with the solids in runoff, an occurrence noted by other researchers (22,23) besides the authors. The probabilistic approach exploits the log-normal distribution of the individual storm data and permits the impact analyst to establish the frequency with which a given pollutant concentration, such as a water quality criterion, would be exceeded in a receiving water as a result of highway runoff. A similar technique was developed by the U.S. Environmental Protection Agency (24) for assessing the effects of urban runoff. It also was recommended by Loftis et al. (25) in a general review of statistical models that might be applied in water quality regulation.

GENERAL IMPACT ASSESSMENT PROTOCOL

Basic Principles

Development of the protocol for assessing the impacts of operating highways on aquatic ecosystems was based on a number of principles as follows:

1. The approach should be hierarchical, so that cases that have different potentials for aquatic ecosystem impact can be distinguished and resources for problem assessment and solution can be proportionately allocated.
2. The protocol should be adaptable for use in different locations through the application of specific analytical procedures that are appropriate to each region.
3. Assessment of the effects of routine highway operations and maintenance procedures should be separate, given the extensive spatial and temporal variability of the latter, even within the same region. Accidental occurrences also should receive separate attention.
4. The protocol should permit assessment of both short- and long-term effects.
5. The highway impact on the receiving water can be assessed most realistically in the context of the aggregate burden created by all activities in the watershed.
6. The methodology should incorporate decision criteria to assist the analyst in forecasting impacts and determining the need for the mitigation of potential impacts.

With regard to the first principle, the devised protocol has a hierarchy of three levels. The first screens out those cases that on the basis of objective criteria, almost certainly would not create significant aquatic impacts under the conditions of routine operation. Those cases that exhibit the potential to create significant impacts are analyzed in full for a typical annual cycle in the second level. In the third level, a more thorough analysis of these potential problems is emphasized and the recommendation is made to consider mitigation measures. After each level, the analyst is directed to assess the effects of maintenance and special problems in accordance with the third principle.

Both short-term occurrences (elevated stream flow and acute pollutant concentrations) and long-term conditions (cumulative pollutant loadings) may create significant aquatic impacts. Criteria for assessing the extent of these impacts are incomplete, however. Although a substantial amount of research has established the responses of aquatic organisms to concentrations of numerous pollutants under test conditions, different conditions of exposure exist in natural waters. Moreover, the biological significance of pollutant loadings is poorly understood in most cases. Given these uncertainties in the face of the need to make judgments about potential environmental impact, the most reasonable procedure appears to be evaluating highway impacts with reference to pre-existing receiving water conditions, as governed by the totality of occurrences in the watershed. In the absence of objective criteria, it can be said, for instance, that the highway can raise stream peak discharge and annual pollutant loadings by certain percentages or increase the frequency of violating a water quality criterion by a particular amount. This strategy is not perfect, because judgment still must be rendered on whether the estimated increases are excessive. Furthermore, it does not take explicit account of loading thresholds that may radically change the aquatic habitat to the detriment of the biological communities, a poorly understood phenomenon. Nevertheless, application of this principle represents an advance in organizing and quantifying impact assessment.

Conceptual Framework

Figure 1 shows a flowchart of the general impact assessment protocol. Level I is a rapid screening mechanism that is intended to identify cases that have a significant impact potential for detailed analysis. Those cases that are not expected to create significant impacts in normal operations are evaluated for the effects of nonroutine occurrences, such as maintenance, accidental spills, or other special problems. The set of screening criteria should be appropriate to the locale as well as conservative, so that only those cases that are certain to avoid significant impacts are dismissed from further analysis.

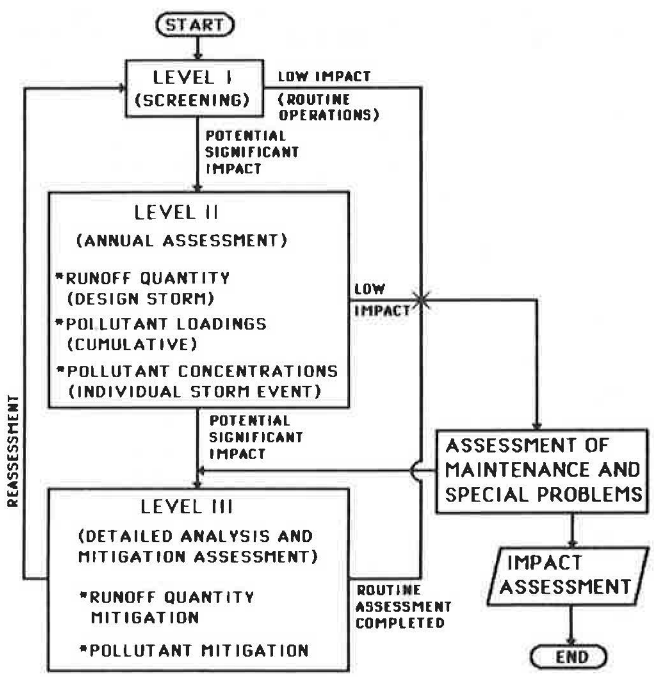


FIGURE 1 Flowchart of general impact assessment protocol.

The purposes of Level II are to guide an impact analyst through assessments of peak stream discharge and annual pollutant load increases that result from the highway's presence, and to evaluate the potential for individual storm events to cause excessive pollution of the receiving water. In each assessment, the contribution of the highway is to be evaluated in the context of pre-existing conditions. Requirements for the analyses include hydrologic, pollutant loading, and pollutant concentration models that are validated for the location of the highway. Hydrologic models of widespread applicability are abundant. However, pollutant load and concentration data are very scattered, and few models are available for any location. The effort of Envirex Inc. (11), cited earlier, has the broadest geographic basis, whereas Shaheen's work (10) was performed in Washington, D.C. Models that stem from the research in California (21) and the Pacific Northwest are available for those regions. The latter model will be discussed in the next section of this paper. Development of specific assessment tools for other areas remains a research need.

Level III is intended to provide further evaluation of the greatest potential routine operating problems revealed by previous analysis and to guide the development of impact mitigation strategies for those problems. It directs the analyst to define more exactly then in Level II, those conditions that were anticipated on the operating highway and in the watershed. In so doing, the closest possible approximation of the degree of impact may be achieved. In this level the recommendation is also made for assessing runoff quantity and quality impacts during critical periods within the annual cycle, if any. Examples are times of maximum flood potential or, conversely, periods of dry weather minimum flows, when the capacity to dilute contaminant concentrations is least.

The guidelines in Level III prompt the impact analyst to consider mitigation such as oil and grease traps, runoff retention/detention (R/D) facilities, and overland runoff discharge through vegetated drainage courses, where the assessment has shown it to be necessary. R/D facilities have the dual advantage of attenuating peak flows and removing some contaminants, especially those in solid form. Various researchers have reported on the efficiency of these facilities in treating stormwater runoff (26-30). Vegetated drainage removes pollutants through settlement, filtration, plant uptake, and various chemical processes and has been tested extensively for municipal and food processing industry wastes (31). The Washington research demonstrated highly efficient TSS and metal removal from highway runoff within 60 m of travel through vegetated ditches (17). Thus, although some data are available to predict R/D device and overland flow performance, the documentation does not extend over a wide range of conditions nor is it sufficient to support the formulation of detailed and generally applicable design criteria. Therefore, highway runoff aquatic impact mitigation represents another major research need.

After consideration of mitigation, the protocol directs the analyst to reevaluate the impact through Levels I and II with the selected mitigation measure in place. When an acceptable anticipated level of impact caused by routine operation is reached, the analyst assesses the potential effects of the maintenance operations and special problems. Included in this analysis are winter sand and deicing agent application, pesticides, accidental spills, and any other features of the highway construction or operation that may affect natural waters beyond routine occurrences. As in other areas of the process, background data and methods of analysis are not well developed in these cases. The analyst is often left with the need to use qualitative or semiquantitative judgment in order to make any assessment. Should the assessment indicate that mitigation of any of these special problems may be required, the analyst is directed back to Level III to develop a management strategy.

AN ILLUSTRATION OF THE IMPACT ASSESSMENT PROTOCOL FOR WASHINGTON STATE

Preparation for the Assessment

The preparation for a complete assessment of the aquatic ecosystem impacts of a highway requires the gathering of substantial data. Some of these data are needed for the quantitative analysis, whereas others serve as background for writing various sections of the EIS. The categories of needed information include general highway design features and operating conditions, drainage system details, physical and hydrologic characteristics of the receiving water, baseline water quality and biological data, and watershed land use characteristics. A list of the specific data needs has been prepared (2) but is not presented here because of its length.

Level I (Screening)

In the version of the protocol developed for Washington, Level I consists of three criteria on which cases can be screened to determine the need for more detailed analysis of routine operating impacts. These criteria concern traffic volume, the proportion of the watershed consumed by the highway, and the availability of mitigation. They are exercised as follows:

1. If all runoff discharges via a vegetated drainage course of at least 60 m in length, go to step 3. Otherwise, proceed to step 2.
2. If projected average daily traffic volume is less than 10,000, proceed to step 3. Otherwise, perform Level II analysis.
3. Determine the total area of the watershed located upstream from the highway runoff discharge point. If there are multiple discharge points, base the determination on the one located farthest downstream.
4. Determine the total area of impervious roadway surface that contributes runoff to the receiving water.
5. If the ratio of impervious roadway surface to total watershed area is less than 0.01, declare no impact from ordinary runoff and proceed to step 6. Otherwise, perform Level II analysis.
6. Analyze impacts associated with the particular anticipated maintenance practices or any special problem areas.

Each stated decision criterion has a basis in the research results. The minimum length of vegetated channel is the length identified (17) as reliably providing 60 to 80 percent reduction of major pollutants in highway runoff. The traffic criterion represents the volume below which no toxic effects appeared in bioassays (18). Concerning the highway-to-watershed area ratio, it is assumed that the runoff is diluted in the receiving stream in approximately the same ratio. Highway runoff can contain concentrations of toxicants comparable to LC_{50}'s (concentration lethal to 50 percent of the organisms in an acute bioassay) (18). A common means of protecting aquatic life is to limit receiving water concentrations to 0.01 x LC_{50}. In addition, investigation of the concentration-probability distributions discussed later in this paper indicates that dilution of 100:1 is generally required to ensure only a slight probability (<0.1 percent) that established water quality criteria will be exceeded. With a high dilution ratio of ordinary runoff and either low traffic volume or drainage over a vegetated drainage course, it can be stated with some assurance that impact of routine operations would be insignificant, and thus more detailed analysis can be avoided.

Level II (Annual Assessment)

The Level II runoff quantity assessment is based on procedures from general practice and the literature because no hydrologic modelling was performed under the research project. In its present form, the guide contains the recommendation to estimate the design for the 25-yr recurrence interval storm according to the Rational Method used by the Washington State Department of Transportation (32). The procedure may be modified by a user to employ a more advanced technique, such as the unit hydrograph or a more advanced hydrologic model. The highway runoff rate should be compared with the existing receiving stream peak discharge for the same design storm condition. This peak discharge may be established through analysis of the gauging record, if a sufficient one exists, by using a distribution model (33). Where there is no adequate gauging record, peak discharge can be estimated from one of a number of hydrologic models or according to a U.S. Geological Survey procedure (34). If the increased discharge caused by highway drainage exceeds a permitted amount or is judged to be excessive, the user is directed to Level III for information on designing detention facilities.

The flowchart in Figure 2 shows a guide to the Level II annual pollutant loading assessment. The flowchart requires that the analyst compare anticipated highway runoff pollutant loadings with loadings that preexisted in the receiving water. These preexisting loadings can be established either through stream water quality and flow data, where they are sufficient, or from the land use characteristics in the watershed. The procedure encompasses discharges to standing bodies of water (lakes and

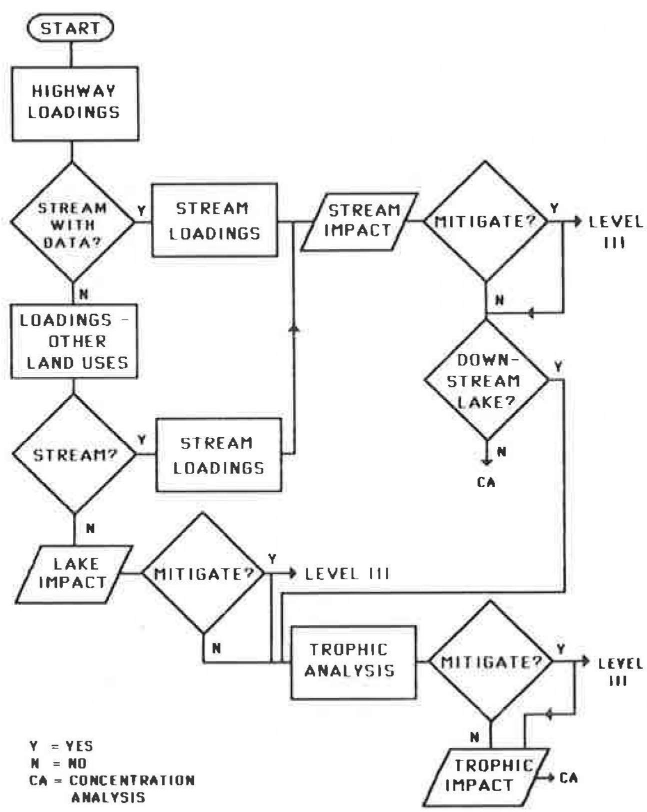


FIGURE 2 Flowchart of level II pollutant loading assessment procedure.

wetlands), along with streams. Its rather arbitrary decision criterion, which determines whether further analysis is recommended, is a loading increase of more than 10 percent of any pollutant in the receiving water as a result of the highway.

In applying this component of the procedure, TSS-loading is first estimated from the cumulative loading model developed from the research data. The basis for this model has been presented elsewhere (13,14,19,20) and represents data from more than 500 storms monitored at nine locations throughout the state. It is expressed as follows:

$$\text{TSS loading} = (K)(VDS)(RC) \tag{1}$$

where

TSS loading = annual mass flux,
K = proportionality constant,
VDS = vehicles travelling during storm periods on an annual basis, and
RC = average site runoff coefficient.

Data analysis yielded different proportionality constants for eastern and western Washington:

Eastern Washington: K (± 1 standard error) = 7.4 (±0.56) kg TSS/highway km/1,000 VDS (2)

Western Washington: K (± 1 standard error) = 1.8 (±0.24) kg TSS/highway km/1,000 VDS (3)

The elevated constant in the former case presumably resulted from deposition of the loose soils of the arid and semiarid region on roadways by relatively high and continuous winds. VDS may be determined from average daily traffic (ADT) records or projections and precipitation duration data that were assembled for a number of locations in the state. In the absence of on-site data, many hydrology texts and handbooks can provide estimates of runoff coefficients for different configurations.

The next step in the annual loading analysis is to modify TSS loading to reflect any runoff treatment provided. Table 1 presents approximate pollutant reduction capacities of various lengths of vegetated channel derived from the research results (17). Contaminant removal by R/D devices can be estimated from the references cited earlier.

Assessment of highway contaminant loadings is completed by applying appropriate multipliers to the TSS loading to estimate annual loadings of other quantities. Table 2 lists those multipliers derived from the Washington State data. They are constants throughout the state for organics and nutrients and linear functions of average daily traffic (ADT) for three heavy metals.

With the completion of the analysis of highway runoff pollutant loadings, the next step of the assessment is to compare these loadings with those that have already occurred in the receiving water. Preexisting loadings may be estimated from stream water quality and flow data, if they are adequate, or published loadings from the various land uses in the watershed for lakes and wetlands and inadequately documented stream cases. Given that consistent units are maintained, the annual pollutant loading in a stream can be estimated as the product of the average discharge and the mean contaminant concentration. For standing water or where hydrologic and water quality data are lacking, export from general land use categories may be estimated from information taken from the literature (tabulated in Table 3) and added to known point source loadings to obtain total loadings. The use of these data instead of stream records is substantially less satisfactory because of the evident dispersion created by aggregating results from many locations.

An additional eutrophication assessment procedure is given in Level II to be employed when the receiving water is a lake. This procedure is based on the phosphorus loading criteria presented by Vollenweider and Dillon (43).

The final component of Level II is an assessment of pollutant concentrations on the basis of an individual storm event. The objective of this assessment is to estimate the probability that a selected pollutant concentration, such as a water quality criterion, will be exceeded in any given storm, from which the frequency of exceedance can be forecast. A series of probability graphs was prepared from the research data to perform the assessment for Washington cases. Data from each sampling station were analyzed to determine the distributions of observed concentrations by using cumulative frequency and histogram graphics. These plots suggested that log-normal distributions would adequately describe the data for each contaminant at each site. The sites

TABLE 1 Highway Runoff Treatment Efficiencies of Various Lengths of Vegetated Channel

Length (m)	Approximate Fraction of Pollutant Remaining
<10	1.00
10-20	0.50
20-30	0.40
30-40	0.30
40-50	0.26
50-60	0.23
>60	0.20

TABLE 2 Multipliers to Estimate Loadings of Other Pollutants from TSS Loading

Pollutant	Multiplier	R^{2a}	Specifications
Solids	0.2	-	For all sites
Chemical oxygen demand	0.4	-	For all sites
Lead	$1.5 \times 10^{-4} + (8.7 \times 10^{-8})$ (ADT)	0.978	West Washington sites
	$5.3 \times 10^{-4} + (2.8 \times 10^{-8})$ (ADT)	0.996	East Washington sites
Zinc	$1.5 \times 10^{-4} + (3.0 \times 10^{-8})$ (ADT)	0.864	West Washington sites
	$2.0 \times 10^{-4} + (3.2 \times 10^{-7})$ (ADT)	0.932	East Washington sites
Copper	$7.9 \times 10^{-5} + (2.7 \times 10^{-9})$ (ADT)	0.739	For all sites
Total Kjeldahl nitrogen	2.7×10^{-3}	-	West Washington sites
	1.2×10^{-3}	-	East Washington sites
Nitrate + nitrite – nitrogen	2.0×10^{-3}	-	For all sites
Total phosphorus	2.1×10^{-3}	-	For all sites

[a]Coefficient of determination for linear regression equations.

TABLE 3 Storm Runoff Pollutant Loadings for General Land Use Categories

	Loading (kg/ha/yr)[a]			
Pollutant	General Urban	General Residential	General Agricultural	Forested or Open
Total suspended solids	450	420	20,100-49,400	7.0-8.5
Chemical oxygen demand	20-270	30-300	NA	2.0
Lead	0.15-0.50	0.06	0.002-0.08	0.01-0.03
Zinc	0.34-0.56	0.02	0.004-0.34	0.01-0.03
Copper	0.04-0.13	0.03	0.002-0.09	0.02-0.03
Nitrate-nitrite-nitrogen	0.34-4.50	0.34-3.8	0.34-8.0	0.34-0.56
Total Kjeldahl nitrogen	8.0	6.1	0.34-34	1.7-3.0
Total phosphorus	2.0	1.8	0.11-9.0	0.07-0.09

Note: NA = not available.

[a]Means given where available; otherwise, ranges are reported *(35-42)*.

were grouped into eastern and western Washington and high- and low-traffic categories for further analysis. Traffic groupings represented the following ADT (all unidirectional): western Washington, high-traffic: 42,000 to 53,000, low-traffic: 7,700 to 8,600; eastern Washington, high-traffic: 17,300, low-traffic: 2,000 to 2,500.

Probability distributions of each pollutant concentration in each group were graphed on log-probability paper. These plots represent the probability of exceeding any given concentration in any storm for the underlying conditions. The graphic representations were used as qualitative tests of log-normality. Log-normal data describe a straight line on such a plot. Figure 3 presents a typical graph in this series. High-traffic, low-traffic, and combined plots generally were linear; the specific traffic-level cases usually provided better fits.

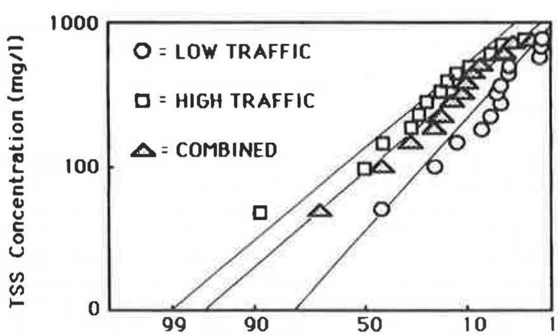


FIGURE 3 Probability distribution of TSS concentration for western Washington sites.

As the final step in the concentration-probability analysis, the distributions for each contaminant were plotted separately for eastern and western Washington high- and low-traffic cases. Parallel curves were added to these graphs to represent pollutant reductions of various amounts. These reductions could be achieved by runoff treatment, dilution by receiving waters, or a combination of the two. When available, water quality criteria were added to the graphs to serve as a basis for judgment of effect and assessment of impact.

Figure 4 provides an example of such a plot. As an illustration of its potential use in impact assessment, suppose untreated highway runoff drains to a stream that provides 25 percent pollutant reduction through dilution and that has a total hardness of 50 mg/L as $CaCO_3$. The probability of exceeding the maximum Pb concentration permitted for the protection of aquatic life (45 Code of Federal Regulations 79318-79379, November 28, 1980) would be 66 percent (i.e., a violation would be expected in two out of every three storms. Should 90 percent Pb reduction be achieved, however, the probability of exceeding the criterion would drop to 0.035 percent, a frequency of violation equivalent to about one storm in 2,900.

Level III (Detailed Analysis and Mitigation Assessment)

Level III has an arrangement parallel to Level II in that it contains guidelines on runoff quantity assessments, accumulated pollutant loadings, and individual event occurrences. The particular procedure would be applied, however, only for the specific problem or problems that are identified in the Level II analysis to have potential significance.

Level III differs from the previous level in several ways. First, the quantity assessment emphasizes design, or redesign, of detention facilities by using customary highway design procedures to prevent excessive stream peak flow increase. The loading assessment is for the monthly period that represents a critical high or low flow condition, rather than annually as in Level II. It also employs a more detailed definition of land use along with the pollutant yields presented in Table 4. Otherwise, the loading analysis is identical to the Level II procedure.

The individual event assessment is directed at water quality impact mitigation and provides a basis to design control facilities. That basis is presented in Figure 5 in the form of a probability distribution of TSS loading for western Washington (an analogous plot exists for eastern Washington). The analyst may select a design probability (e.g., the loading exceeded in only 10 percent of the storms) to use in selecting and sizing the control device. Pollutants other than TSS may be brought into the analysis by using the multipliers in Table 2.

Assessment of Impacts Associated with Maintenance Practices and Special Problem Areas

The assessment methodology described heretofore applies only to the aquatic impacts associated with ordinary runoff events on normally operating highways. Periodic and extraordinary phenomena must be analyzed separately. Included in this category are winter sanding and deicing, pesticide application, construction practices that create continuing effects on surface waters, and accidental spills. The

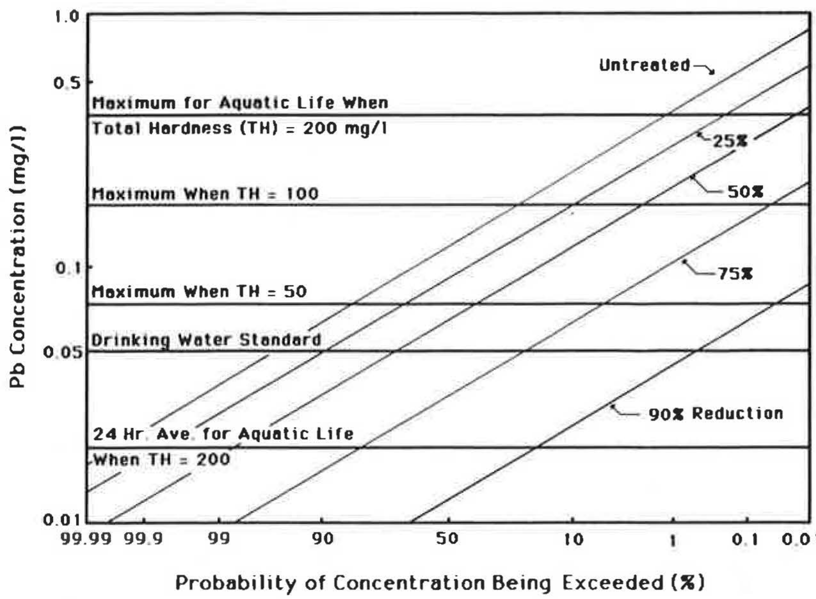


FIGURE 4 Probability distribution of lead concentration for western Washington low-traffic cases.

TABLE 4 Storm Runoff Pollutant Loadings for Specific Land Use Categories

Pollutant	Loading (kg/ha/yr)[a] Central Business District	Other Commercial	Industrial	Single-Family Residential	Multiple-Family Residential	Cropland	Pasture	Forested	Open
Total suspended solids	1,080	840	56	17	440	450	340	85	6.7
Chemical oxygen demand	1,070	1,020	63	28	330	NA	NA	NA	2.0
Lead	7.1	3.0	1.0-7.1	0.11	0.67	0.005-0.006	0.003-0.015	0.01-0.03	NA
Zinc	3.0	3.3	3.5-12	0.22	0.34	0.03-0.08	0.02-0.17	0.01-0.03	NA
Copper	2.1	NA	0.34-1.1	0.03	0.34	0.01-0.06	0.02-0.05	0.02-0.03	NA
Nitrate-nitrite-nitrogen	4.5	0.67	0.45	0.34	3.8	7.9	0.34	0.56	0.34
Total Kjeldahl nitrogen	15	15	2.2-15	1.1-5.6	3.4-4.5	1.7	0.67	2.9	1.7
Total phosphorus	2.8	2.7	0.90-4.0	0.22-1.5	1.3-1.6	0.34	0.07	0.09	0.07

Note: NA = not available.

[a]Means given where available; otherwise, ranges are reported *(35-42)*.

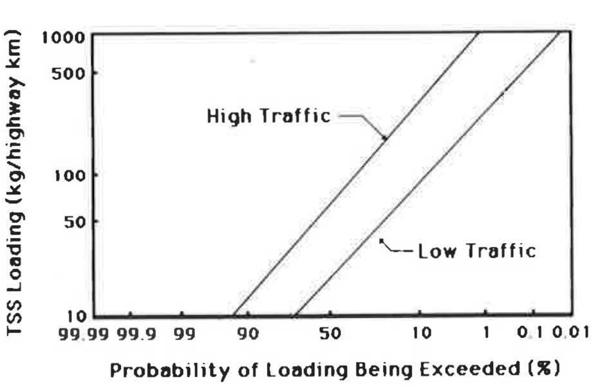


FIGURE 5 Probability distribution of TSS loading for western Washington.

assessment protocol provides guidance in these areas, although without the specificity made possible by the large data base that underlies the routine evaluation (Levels I-III).

The various pollutant loadings are augmented by the contribution from sand. Winter data demonstrated that sanding contributed a major portion of TSS seasonally, which varied with the sand application rate and other sources of solids (44). The results were insufficient to model the proportion of applied sand that entered the runoff, however; consequently, it is necessary to roughly estimate the proportion on the basis of sand characteristics, plowing, and sweeping.

The Washington research demonstrated that the ratios of other pollutants to TSS associated with sanding were equal to those reported in Table 2 at the high-traffic sites. With less traffic, pollutant deposition failed to saturate the sand particles, and the ratios were substantially lower on a cumulative basis (44). It is thus recommended that the loadings of other pollutants be established according to the procedures given for Levels II and III when ADT is projected to exceed 10,000. With less traffic, the assessment should reflect the elevated TSS loading as a result of sanding but should not augment the loadings of other pollutants in proportion to sanding TSS.

Deicing impacts were not specifically investigated in the Washington State research. The guide does provide a procedure drawn from work in Massachusetts (45) for estimating sodium and chloride loadings and concentrations from prevailing application rates.

A comprehensive study of leachates from woodwaste fill sections was undertaken during the research (15). The protocol included an aquatic impact assessment procedure for that regionally important problem. Pesticide applications and accidental spills were covered qualitatively. Insufficient risk data exist to relate, in a general fashion, the occurrence of spills with highway characteristics. In a specific case, data may be available from the same or a similar highway that would enable the analyst to make some estimates of accident probabilities, and some relevant reports were cited for the analyst's consideration (46,47). The risk of impacts to aquatic systems by accidental spills is greatest with uninterrupted transport to the water body, which allows little opportunity for removal of toxicants or time for reaction (application of spill management techniques).

SUMMARY AND CONCLUSIONS

Assessment of the impacts on aquatic ecosystems of operating highways is in its infancy. Organized paradigms to guide the assessment have been lacking, and analytical tools for quantification of anticipated impacts are few in number and are generally validated for limited areas. The same could be said of the state of impact assessment in many other fields. A general protocol has been proposed to fill the former need. The protocol contains a hierarchical arrangement to identify the most serious cases for the most complete analysis and allocation of resources for mitigation. The protocol also contains a recommendation for the evaluation of the effects of the highway in the context of other activities that influence the runoff receiving water, and for the consideration of both short- and long-range potential impacts.

A large data base was analyzed to develop the specific analytical procedures required for the application of the protocol to assess the impacts of operating highways in the state of Washington. The procedures include a simple cumulative pollutant loading model, a probabilistic method of evaluating potential acute effects of a single storm, hydrologic assessment techniques drawn from the literature, and semiquantitative or qualitative means of analyzing the potential effects of nonroutine occurrences, such as intermittent maintenance operations, accidents, and other special problems. Because of similarities in geomorphology, climate, overall land use, and aquatic ecosystems, it is the authors' opinion that these procedures are also applicable in northern California, Oregon, and portions of Idaho and British Columbia. Moreover, they offer an example of methodology that could be developed for the assessment of highway impacts in other locations or for conducting objective and quantitative environmental assessments in many other situations.

The results of the Washington highway runoff research was that limited problem areas were identified, thus providing a basis for the reduction of mitigation costs overall and the application of resources to those cases most in need of attention. Adequate research would permit the application of this principle elsewhere, thereby achieving savings and reduced legal challenges while providing environmental protection where the needs are greatest.

ACKNOWLEDGMENTS

The authors wish to acknowledge the sponsorship of the Washington State Department of Transportation and the FHWA for the research underlying this paper. Support provided for preparation of the paper by the Washington State Transportation Center and G. Scott Ruterford is also gratefully acknowledged.

REFERENCES

1. Guidelines for Review of Environmental Impact Statements, Vol. I: Highway Projects (draft). Office of Federal Activities, U.S. Environmental Protection Agency, 1978.
2. R.R. Horner and B.W. Mar. Guide for Water Quality Impact Assessment of Highway Operations and Maintenance. Report FHWA WA-RD-39.14. Department of Civil Engineering, University of Washington, Seattle, Wash., 1982, 81 pp.
3. R.R. Horner, B.W. Mar, B.L. Chaplin, and F.E. Conroy. A Plan for the Implementation of Highway Runoff Water Quality Research Results. Department of Civil Engineering, University of Washington, Seattle, Wash., 1983, 9 pp.
4. J.D. Sartor and G.B. Boyd. Water Pollution Aspects of Street Surface Contaminants. Report EPA-R2-72-081. U.S. Environmental Protection Agency, 1972, 236 pp.
5. R.E. Pitt and G. Amy. Toxic Materials Analysis of Street Surface Contaminants. Report EPA-R2-73-283. U.S. Environmental Protection Agency, 1973.
6. R.O. Sylvester and F.B. DeWalle. Character and Significance of Highway Runoff Waters. Department of Civil Engineering, University of Washington, Seattle, Wash., 1972, 97 pp.
7. G. Soderlund and H. Lehtinen. Comparisons of Discharges from Urban Storm Water Runoff, Mixed Storm Overflow and Treated Sewage. Proc., 6th International Conference on Water Pollution Research, Pergamon Press, Ltd., London, England, 1972, 10 pp.
8. Municipality of Metropolitan Seattle. Freeway Runoff from the I-90 Corridor. Washington State Highway Commission, Seattle, Wash., 1973, 75 pp.
9. Municipality of Metropolitan Seattle. Water Quality Investigations Along SR-90, Phase IV. Washington State Highway Commission, Seattle, Wash., 1973, 52 pp.
10. D.G. Shaheen. Contributions of Urban Roadway Usage to Water Pollution. Report EPA-600/2-75-004. U.S. Environmental Protection Agency, 1975, 350 pp.
11. N.P. Kobriger, T.L. Meinholz, M.K. Gupta, and R.W. Agnew. Constituents of Highway Runoff, Vol. III: Predictive Procedure for Determining Pollutant Characteristics in Highway Runoff. Report FHWA/RD-81/044. Envirex, Inc., Milwaukee, Wis., 1981, 197 pp.
12. D.L. Clark, R.L. Asplund, J.F. Ferguson, and B.W. Mar. Composite Sampling of Highway Runoff. Journal of the Environmental Engineering Division, ASCE, Vol. 107, No. 5, 1981, pp. 1067-1081.
13. R.L. Asplund, J.F. Ferguson, and B.W. Mar. Characterization of Highway Runoff in Washington State. Journal of the Environmental Engineering Division, ASCE, Vol. 108, No. 2, 1982, pp. 391-404.
14. T.W. Chui, B.W. Mar, and R.R. Horner. A Pollutant Loading Model for Highway Runoff. Journal of the Environmental Engineering Division, ASCE, Vol. 108, No. 6, 1982, pp. 1193-1210.
15. K.H. Vause, J.F. Ferguson, and B.W. Mar. Water Quality Impacts Associated with Leachates from Highway Woodwaste Embankments. Report FHWA WA-RD-39.1. Department of Civil Engineering, University of Washington, Seattle, Wash., 1980, 34 pp.
16. K.R. Zawlocki, J.F. Ferguson, and B.W. Mar. A Survey of Trace Organics in Highway Runoff in Seattle, Washington. Report FHWA WA-RD-39.9. Department of Civil Engineering, University of Washington, Seattle, Wash., 1981, 44 pp.

17. T.S. Wang, D.E. Spyridakis, B.W. Mar, and R.R. Horner. Transport, Deposition and Control of Heavy Metals in Highway Runoff. Report FHWA WA-RD-39.10. Department of Civil Engineering, University of Washington, Seattle, Wash., 1982, 33 pp.
18. G.J. Portele, B.W. Mar, R.R. Horner, and E.B. Welch. Effects of Seattle Area Highway Stormwater Runoff on Aquatic Biota. Report FHWA WA-RD-39.11. Department of Civil Engineering, University of Washington, Seattle, Wash., 1982, 45 pp.
19. B.W. Mar, R.R. Horner, J.F. Ferguson, D.E. Spyridakis, and E.B. Welch. Summary--Washington State Highway Runoff Water Quality Study, 1977-1982. Report FHWA WA-RD-39.17. Department of Civil Engineering, University of Washington, Seattle, Wash., 1982, 118 pp.
20. L.M. Little, R.R. Horner, and B.W. Mar. Assessment of Pollutant Loadings and Concentrations in Highway Stormwater Runoff. Report FHWA WA-RD-39.12.1. Department of Civil Engineering, University of Washington, Seattle, Wash., 1983, 41 pp.
21. J.A. Racin, R.B. Howell, G.R. Winters, and E.C. Shirley. Estimating Highway Runoff Quality. Report FHWA-CA-RL-82-11. California Department of Transportation, Sacramento, 1982.
22. A.S. Donigian, Jr., and N.H. Crawford. Simulation of Nutrient Loadings in Surface Runoff with the NPS Model. Report EPA-600/3-77-065. Hydrocomp, Inc., Palo Alto, Calif., 1977.
23. S.W. Zison. Sediment-Pollutant Relationships in Runoff from Selected Agricultural, Suburban and Urban Watersheds. Report EPA-600/3-80-22. Tetra Technology, Inc., Lafayette, Calif., 1980, 135 pp.
24. Preliminary Results of the Nationwide Urban Runoff Program. Vol. 1. Water Planning Division, U.S. Environmental Protection Agency, 1982.
25. J.C. Loftis, R.C. Ward, and G.M. Smillie. Statistical Models for Water Quality Regulation. Journal of the Water Pollution Control Federation, Vol. 55, No. 8, 1983, pp. 1098-1104.
26. W. Whipple and J.V. Hunter. Settleability of Urban Runoff Pollution. Journal of the Water Pollution Control Federation, Vol. 53, No. 12, 1981, pp. 1726-1731.
27. C.W. Randall, K. Ellis, T.J. Grizzard, and W.R. Knocke. Urban Runoff Pollution Removal by Sedimentation. Presented at Conference on Stormwater Detention Facilities, New England College, Henniker, N.H., Aug. 1982.
28. W.J. Davis, R.H. McCuen, and G.E. Kamedulski. The Effect of Stormwater Detention on Water Quality. Presented at International Symposium on Urban Stormwater Management, University of Kentucky, Lexington, Ky., July 1978.
29. D.C. Curtis and R.H. McCuen. Design Efficiency of Stormwater Detention Basins. Journal of Water Resources Planning Management Division, ASCE, Vol. 103, No. 1, 1977, pp. 125-140.
30. R.H. McCuen. Water Quality Trap Efficiency of Storm Water Management Basins. Water Resources Bulletin, Vol. 16, No. 1, 1980, pp. 15-21.
31. D.J. Hinrichs, J.A. Faisst, and D.A. Pivetti. Assessment of Current Information on Overland Flow Treatment of Municipal Wastewater. Report EPA-430/9-80-002. Office of Water Programs Operations, U.S. Environmental Protection Agency, 1980.
32. Highway Hydraulic Manual. Washington State Highway Commission, Washington State Department of Transportation, Olympia, Wash., 1972.
33. R.K. Linsley, Jr., M.A. Kohler, and J.L.H. Paulhus. Hydrology for Engineers. 2nd ed. McGraw-Hill Book Co., Inc., N.Y., 1975, 482 pp.
34. J.E. Cummans, M.R. Collings, and E.G. Nassar. Magnitude and Frequency of Floods in Washington. Open-File Report 74-336. U.S. Geological Survey, Tacoma, Wash., 1975.
35. S.R. Wiebel, R.J. Anderson, and R.L. Woodward. Urban Land Runoff as a Factor in Stream Pollution. Journal of the Water Pollution Control Federation, Vol. 36, 1964, pp. 914-924.
36. Storm Water Pollution from Urban Land Activity. Avco Economic Systems Corporation, Washington, D.C., 1970.
37. Environmental Management for the Metropolitan Areas, Cedar-Green River Basins, Washington, Part II: Urban Drainage, Appendix C: Stormwater Monitoring Program. U.S. Army Corps of Engineers, Seattle, Wash., 1974.
38. J.P. Heaney, W.C. Huber, and S.J. Nix. Storm Water Management Model, Level I, Preliminary Screening Procedures. Report EPA-600/2-76-275. Department of Environmental Engineering Sciences, University of Florida, Gainesville, 1976.
39. D.L. Reese. Urban Stormwater Runoff Guide, Boise Valley, Idaho. U.S. Army Corps of Engineers, Walla Walla, Wash., 1976.
40. J.M. Omernik. Non-Point Source-Stream Nutrient Level Relationships: A Nationwide Survey. Report EPA-600/3-77-105. U.S. Environmental Protection Agency, Corvallis, Oreg., 1977.
41. M.P. Wanielista. Stormwater Management, Quantity and Quality. Ann Arbor Science Publishers, Inc., Ann Arbor, Mich., 1978.
42. F.X. Browne and T.J. Grizzard. Non-Point Sources. Journal of the Water Pollution Control Federation, Vol. 51, No. 6, 1979, pp. 1428-1444.
43. R.A. Vollenweider and P.J. Dillon. The Application of the Phosphorous Loading Concept to Eutrophication Research. Report NRCC 13690. Center for the Environment, Burlington, Ontario, Canada, 1974.
44. R.L. Asplund. Characterization of Highway Stormwater Runoff in Washington State. M.S.E. thesis. Department of Civil Engineering, University of Washington, Seattle, Wash., 1980, 186 pp.
45. L.R. Frost, Jr., S.J. Pollock, and R.F. Wakelee. Hydrologic Effects of Highway Deicing Chemicals in Massachusetts--Executive Summary. Open-File Report 81-210. U.S. Geological Survey, Boston, Mass., 1981.
46. P.D. Eagen. Views of Risk and the Highway Transportation of Hazardous Materials. Report FHWA WA-RD-39.8. Department of Civil Engineering, University of Washington, Seattle, Wash., 1980, 44 pp.
47. W.B. Andrews, R.E. Rhoads, A.L. Franklin, B.M. Cole, and R.G. Rau. Hazardous Material Transportation Risks in the Puget Sound Region. Battelle Pacific Northwest Laboratories, Richland, Wash., 1981.

Consequential Species of Heavy Metals in Highway Runoff

Y. A. YOUSEF, H. H. HARPER, L. P. WISEMAN, and J. M. BATEMAN

ABSTRACT

Speciation of heavy metals in aquatic systems plays a key role in their transport, chemical reactions, and bioavailability. Those physical and chemical forms that may cause significant consequences, known as consequential species, should be identified before the potential environmental impact of the metal can be determined. Species of dissolved lead, zinc, copper, and cadmium were identified by using anodic stripping voltametry for rainfall, highway and bridge runoff, and receiving streams at the intersections of Maitland Interchange and I-4, and US-17-92 and Shingle Creek in the central Florida area. Natural water systems reduce ionic species by complexation of incoming trace metals, which results in the reduction of their toxic effects. Most of the heavy metals in highway runoff that are discharged into detention/retention ponds similar to the Maitland site are concentrated in the upper layer (approximately 5 to 6.8 cm) of the bottom sediments. The potential for their release is unlikely if an aerobic environment of the sediment is maintained.

Highway stormwater runoff contains significantly higher concentrations of trace metals, particularly lead (Pb), zinc (Zn), iron (Fe), cadmium (Cd), chromium (Cr), nickel (Ni), and copper (Cu), than the adjacent water environment (1,2). As these metals reach the ecosystem, they will undergo physical, chemical, and biological transformations. They may be adsorbed on clay particles, taken up by plant and animal life, or remain in solution. Particulate fractions will settle to the bottom sediments, and heavy metals may resuspend or redissolve back into solution if the environmental conditions permit. Fate and transformations of trace elements in natural environments follow complex processes and much information is needed before their impact can be predicted.

Environmental scientists realize that the total concentration of a particular metal in natural waters can be very misleading. A water with high total metal concentration may be, in fact, less toxic than another water with a lower concentration of different forms of that metal. For example, ionic Cu is far more toxic toward aquatic organisms than organically bound Cu. Also, biotoxicity of Cu complexes decreases as their stability increases. Therefore, it is of prime importance to fully understand different dissolved metal species in an aqueous environment and to study the impact of those species on existing biota.

Information on consequential species of metals in highway runoff after their introduction into aquatic systems is essential to enhance the understanding of their impact and better evaluate measures for their control. This paper contains a summary of the results obtained from the following field and laboratory investigations:

1. Analysis of major constituents and trace metals in water samples of rainfall, stormwater, and water from a detention pond that receives highway and bridge runoff, and Shingle Creek, which flows beneath that bridge;
2. Partitions of trace metals between the bottom sediments and overlying water column;
3. Biotoxicity for various metal species; and
4. Speciation of trace metals in natural water by using a computerized chemical model (WATEQ2).

STUDY SITES

Two sites were selected to investigate the consequential species of heavy metals in highway stormwater runoff: (a) the intersection of Interstate 4 and Maitland Interchange and (b) the intersection of US-17-92 and Shingle Creek, south of Kissimmee, Florida. The traffic on I-4 at the Maitland site exceeds 50,000 vehicles per day and the traffic on US-17-92 at the Shingle Creek site exceeds 12,000 vehicles per day. The average daily traffic (ADT) count at each site for the past 3 yr was provided by David Harrell of the Florida Department of Transportation (FDOT) in personal communication in 1984, and is presented in Table 1.

TABLE 1 Daily Traffic Count for Selected Sites to Study Consequential Species

		Average Daily Traffic		
Site Location	Traffic Lanes	1981	1982	1983
NE I-4 and Maitland	Eastbound	36,013	38,717	51,454
Interchange	Westbound	35,430	39,288	50,023
SW I-4 and Maitland	Eastbound	45,207	47,456	54,482
Interchange	Westbound	43,705	50,008	52,810
Maitland Avenue at	Eastbound	12,506	14,305	15,833
SR 427	Westbound	12,896	13,965	15,683
US-17-92 and Shingle Creek, St. #3, SW Kissimmee City Limit	Combined	12,856	12,117	12,254

Stormwater runoff from I-4 is delivered by overland flow over grassy swales to storm drain inlets or detention ponds A, B, and C (Figure 1). Stormwater runoff from the Maitland Boulevard bridge that crosses over I-4 is conveyed directly off the roadway surface through stormwater inlets to culverts that ultimately discharge into Pond A. The ponds are interconnected and the water from the west pond flows over a wooden weir at its southern end, which is connected to Lake Lucien by means of a culvert and a short, densely vegetated ditch (Figure 1). The west pond is triangular in shape, with a surface

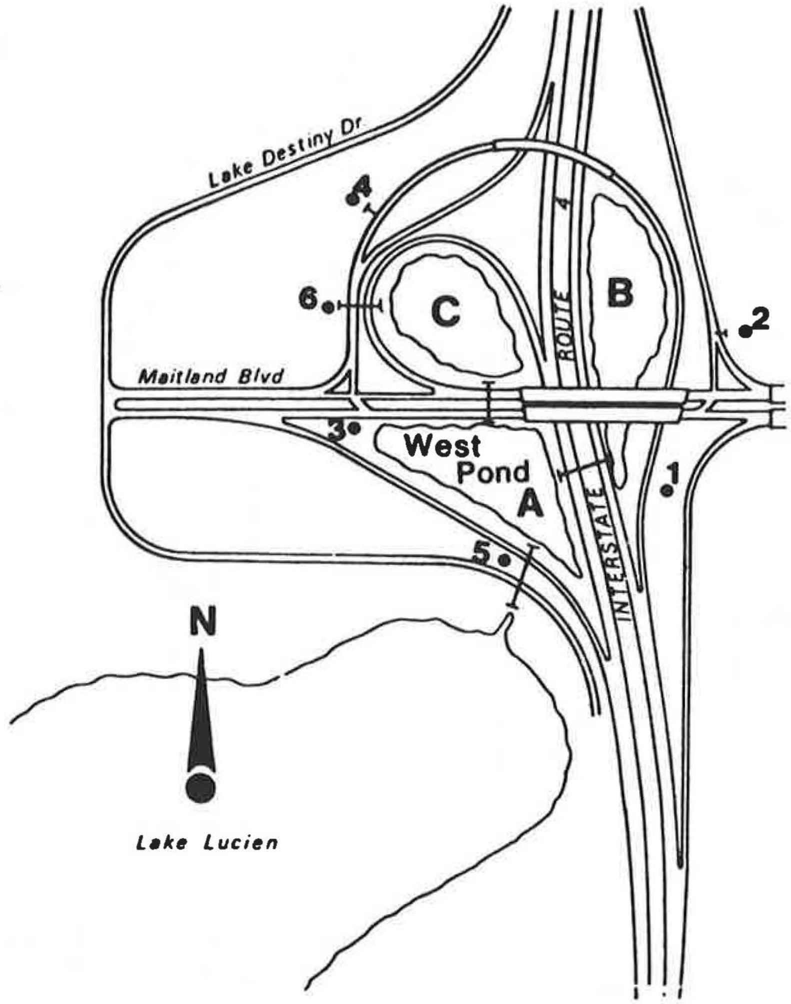


FIGURE 1 Sampling site for highway runoff at Maitland Interchange and Interstate 4.

area of approximately 1.2 ha (3 acres) and a depth of 1.5 to 2 m.

US-17-92 crosses Shingle Creek south of the Kissimmee city limit, approximately 1 mile from Lake Tohopekaliga. The roadway is a two-lane undivided highway with a 1983 ADT count of 12,250 vehicles. Stormwater runoff is removed from the bridge area by a system of numerous 10-cm scupper drains that empty onto the underlying wetland areas (Figure 2).

FIELD AND LABORATORY PROCEDURES

Water samples were collected for heavy-metal analysis from the west pond at the Maitland Interchange and from the surrounding drainage area as shown in Figure 1. Sampling locations, shown in Figure 2, were selected beneath the bridge from the scupper drains (S-1) and from the Shingle Creek water near the bridge site (S-2) to study metal speciation in the highway bridge runoff and receiving stream.

Water samples were also collected from rainfall, highway runoff, and the detention pond at the Maitland Interchange and I-4 intersection for analysis of particulate and dissolved metal content. Similar analyses were performed on water samples collected from highway bridge runoff and the receiving stream at US-17-92 and Shingle Creek. Various metal species of Pb, Zn, Cu, and Cd in solution were determined by using anodic stripping voltametry (ASV) techniques that followed the proposed scheme by Bately and Florence (3). These metals are the most abundant in highway runoff and received the most study (4). Together, these accounted for approximately 90 to 98 percent of the total metals observed, with Pb and Zn accounting for 89 percent. Speciation of other metals by using the same technique has not been fully developed. However, available computer programs may assist in the determination of various species of additional metals that exist under a known set of environmental parameters (5).

A detailed discussion of site description, sampling collection, and utilized procedures is presented in a recent report submitted to the Florida Department of Transportation (FDOT) (6).

WATER ANALYSIS

Water samples were collected in duplicate, filtered in the field, and stored inside ice-packed chests. Five sets of the water samples, collected on April 6, May 13, June 7, July 5, and August 1, 1983, were analyzed by both the U.S. Geological Survey (USGS) Water Resources Laboratory in Ocala and the Chemistry and Environmental Engineering Laboratories at the University of Central Florida (UCF) in Orlando. The samples were preserved and stored in the refrigerator at the laboratory until analysis, as speci-

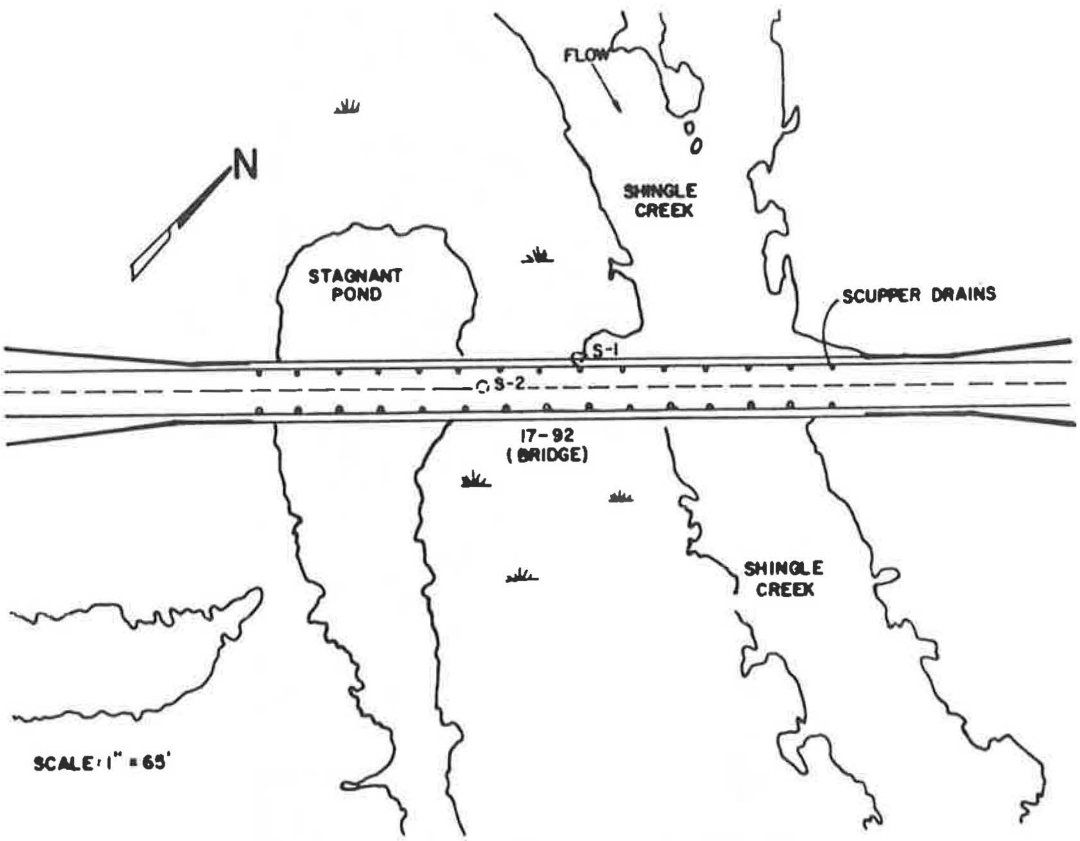


FIGURE 2 Locations of sample sites at US-17-92 and Shingle Creek.

fied in a report by the U.S. Environmental Protection Agency (7). The analysis included major anions, cations, and trace metals. The overall averages of the results from the USGS and UCF analyses are presented in Table 2.

The average pH values from rainfall samples were slightly higher than 5; however, other water samples tested were close to neutral. The acidic rainfall was generally neutralized as it flowed over the drainage basin. Also, runoff water picked up dissolved solids, which was evident from the increase in specific conductance and dissolved solids mea-

TABLE 2 Overall Average Dissolved Water Quality Parameters

	Maitland Interchange						US-17-92 and Shingle Creek			
	Rainfall		Runoff		Pond		Shingle Creek		Runoff	
Parameter	$\bar{X}$	$\bar{\sigma}$	$\bar{X}$	$\bar{\sigma}$	$\bar{X}$	$\bar{\sigma}$	$\bar{X}$	$\bar{\sigma}$	$\bar{X}$	$\bar{\sigma}$
pH (lab)	5.2	0.7	6.9	0.2	6.8	0.2	6.8	0.2	7.2	0.2
Special conditions (μmho/cm)	18.0	4.0	123.0	52.0	186.0	21.0	210.0	64.0	237.0	101.0
Color units	1.0	2.5	10.0	9.0	10.0	7.0	220.0	69.0	50.0	36.0
Dissolved solids	9.8	2.1	75.9	39.9	113.3	10.4	159.9	40.8	173.3	95.4
Alk ($CaCO_3$)	1.0	1.0	44.4	16.2	50.4	9.5	40.4	15.4	74.5	22.1
TH ($CaCO_3$)	4.3	2.1	48.8	25.6	70.8	9.9	60.6	12.6	95.5	49.2
NCH ($CaCO_3$)	3.3	2.2	8.0	6.6	20.2	2.4	20.2	5.3	23.5	24.9
HCO_3	1.5	1.3	54.2	20.2	61.7	12.0	49.2	19.1	91.0	26.9
SO_4^{-2}	2.7	1.4	11.9	10.2	26.6	5.5	23.3	7.0	28.1	30.2
Cl^-	1.6	0.7	2.9	1.5	5.1	1.0	21.8	5.3	10.8	9.3
TH as N	0.66	0.50	0.79	0.18	0.51	0.16	1.37	0.42	3.15	2.16
Organic N as N	0.27	0.13	0.32	0.18	0.38	0.14	1.02	0.45	1.78	1.38
NH_4-N	0.11	0.10	0.09	0.07	0.04	0.02	0.14	0.1–	0.69	1.03
NO_2-N	0.01	0.0	0.02	0.01	0.01	0.0	0.01	0.0	0.07	0.08
NO_3-N	0.31	0.26	0.33	0.26	0.15	0.26	0.26	0.24	0.48	0.77
TP-P	0.02	0.0	0.05	0.01	0.01	0.0	0.22	0.13	0.12	0.03
OP-P	0.01	0.0	0.03	0.02	0.0	0.0	0.22	0.10	0.09	0.04
Ca^{+2}	1.6	1.0	27.0	28.3	20.5	3.4	17.3	4.1	27.8	19.2
Mg^{+2}	0.2	0.1	1.17	1.34	4.5	0.4	4.2	1.0	0.9	0.9
Na^{+2}	2.2	2.6	2.9	2.3	5.6	0.9	17.3	9.9	3.7	2.2
K^+	0.5	0.4	1.7	1.6	4.3	1.6	2.6	1.0	3.9	5.6
SiO_2	0.1	0.1	1.9	1.4	1.2	1.2	4.5	1.7	3.0	0.8
Humic acids	1	0.5	5	4	4	2	18	14	10	9

Note: All concentrations are expressed in milligrams per liter.

surements between rainfall and runoff samples. Rainfall samples averaged approximately 10 mg per liter of dissolved solids, whereas runoff samples averaged between 76 and 173 mg per liter. There was little difference between values measured in runoff water and those measured in receiving water. Dissolved solids concentrations can be expressed in terms of specific conductance. The ratios of dissolved solids concentration to specific conductance averaged 0.54, 0.62, 0.61, 0.76, and 0.73 for rainfall, highway runoff, the Maitland ponds, Shingle Creek water, and bridge runoff, respectively. Water characteristics for the Maitland site showed distinct differences from those for the Shingle Creek site. Also, dissolved solids, alkalinity, and total hardness in the Maitland pondwater were higher than the same parameters in runoff water, presumably because of their concentration by evaporation of the pondwater.

The average total nitrogen (TN) and phosphorus (TP) concentrations in Maitland pondwater were lower than those in rainfall and runoff waters. Inorganic nitrogen was the major component of rainwater and organic nitrogen was the major component of pondwater. The average inorganic nitrogen and total phosphorus concentrations in the Maitland pondwater samples did not exceed 30 percent of the average concentrations in highway runoff water. The pond appeared to be very efficient in the removal of inorganic nitrogen and phosphorus species from highway runoff water. The same conclusions were reached during a detailed analysis of the pondwater (8).

The TN in the bridge runoff water was higher than the TN in Shingle Creek water although the TP was lower. Shingle Creek is a flowing stream that receives municipal wastewater effluent, agricultural runoff, and urban runoff. The creek water is highly colored, averaging 220 color units caused by humic substances from the decay of vegetation. The humic substance concentration averaged 18 mg per liter in Shingle Creek water and 4 mg per liter in the Maitland pondwater. Similarly, silicon dioxide (SiO_2) concentration averaged much higher values in Shingle Creek water than the Maitland pondwater.

The analysis indicated that rainwater washed off deposits on highway surfaces and dissolved the contaminants in stormwater runoff. Major cations, particularly Ca, Mg, Na, and K, were dissolved in surface runoff water. The quality of runoff appeared to be improved by retention/detention in the Maitland pond. The calcium concentration in the pondwater was lower than in runoff water; however, Mg, Na, and K concentrations were higher in the pondwater than in the runoff water. Calcium may be reduced by precipitation and removal from the water column, and other cations are concentrated by evaporation.

METAL SPECIATION IN WATER SAMPLES

The analysis followed the speciation scheme by Bately and Florence (3), which required several treatment steps to separate the various species of labile and nonlabile trace metals. Labile species may include organic colloidal and inorganic soluble and colloidal forms. Also, nonlabile species may include organic soluble and colloidal and inorganic soluble and colloidal forms. The peak heights measured from current (I) versus voltage (E) diagrams for Zn, Cd, Pb, and Cu in water samples before and after Chelex-100 treatment and before and after exposure to ultraviolet light should allow the speciation determination of various metals. The data indicate that the average dissolved concentrations in Maitland rainfall, runoff, and pondwater were 2.49, 1.61, and 1.05 μg of Cd per liter; 8.15, 23.0, and 10.8 μg of Zn per liter; 8.7, 40.7, and 20.4 μg of Pb per liter; and 66.1, 26.6, and 16.6 μg of Cu per liter, respectively. Also, Shingle Creek water and bridge runoff averaged 1.76 and 2.92 μg of Cd per liter; 14.5 and 15.3 μg of Zn per liter; 18.8 and 27.7 μg of Pb per liter; and 8.86 and 18.6 μg of Cu per liter, respectively. The average metal concentrations in Maitland pondwater were lower than those detected in rainfall and runoff water. The pond is efficient in the removal of metals that accumulate in the bottom sediments. The average metal concentrations in the Shingle Creek water were similar in that they were lower than those detected in the highway bridge runoff that crossed over the creek at US-17-92.

The relative distribution of various dissolved species of trace metals that were detected in water samples collected during this study is presented in Table 3. The data showed that labile, organic, and colloidal fractions averaged 82.0, 5.3, and 32.9 percent for Cd; 92.2, 0.3, and 42.7 percent for Zn; 60.9, 22.1, and 55.6 percent for Pb; and 63.7, 48.9, and 69.8 percent for Cu, respectively, in all water samples tested. The organic fraction for dissolved Cu in water samples was the highest among all metals tested. The organic fraction in all metals tested followed a decreasing order: Cu > Pb > Cd > Zn. The labile fraction followed a similar decreasing order: Zn > Cd > Cu > Pb. Also, the colloidal fraction followed a decreasing order of Cu > Pb > Zn > Cd. It can be concluded, then, that Zn and Cd from highway runoff are more reactive in natural environments than Cu and Pb. Zn and Cd, however, may exist in ionic forms and are more readily available to biota in natural systems.

TABLE 3 Relative Distribution for Various Dissolved Species of Trace Metals in Water Samples

		Percentage in Water from				
		Maitland Interchange			US-17-92	
Metal	Form	Rainfall	Runoff	Pond	Bridge Runoff	Shingle Creek
Cd	Labile	85.9	84.7	86.3	78.1	75.2
	Nonlabile	14.1	15.3	13.7	21.9	24.8
	Organic	1.1	4.3	4.2	3.4	13.3
	Inorganic	98.9	95.7	95.8	96.6	86.6
	Colloidal	19.3	36.4	31.7	38.3	38.8
	Noncolloidal	80.7	63.6	68.3	61.7	61.2
Zn	Labile	93.7	92.5	96.3	92.5	89.5
	Nonlabile	6.3	7.5	3.7	7.5	10.5
	Organic	0.0	0.7	0.3	0.3	0.2
	Inorganic	100	99.3	99.7	99.7	99.8
	Colloidal	45.8	23.6	83.7	29.2	31.0
	Noncolloidal	54.2	76.4	16.3	70.8	69.0
Pb	Labile	65.6	72.7	55.4	43.8	67.2
	Nonlabile	34.4	27.3	44.6	56.2	32.8
	Organic	14.6	15.4	17.3	44.0	19.3
	Inorganic	85.4	84.6	82.7	56.0	80.7
	Colloidal	63.3	36.7	54.2	68.7	54.9
	Noncolloidal	36.7	63.3	45.8	31.3	45.1
Cu	Labile	84.0	45.9	81.0	58.7	49.0
	Nonlabile	16.0	54.1	19.0	41.3	51.0
	Organic	38.3	56.6	53.8	33.4	62.2
	Inorganic	61.7	43.4	46.2	66.6	37.8
	Colloidal	59.8	75.6	72.1	62.0	79.7
	Noncolloidal	40.2	24.4	27.9	38.0	20.3

SPECIATION OF TRACE METALS IN SEDIMENTS

Concentrations of trace metals measured in the incoming highway runoff appear to exist predominantly in association with particulate matter. Particulate fractions accounted for 42 percent of the total Cd, 86 percent of the total Zn, 47 percent of the total

TABLE 4 Comparison of Average Heavy-Metal Concentrations in Stormwater Runoff and in the Retention Basin (West Pond) at Maitland Interchange

	Average Incoming Stormwater Quality (N=16)			Average Retention Basin Water Quality (N=34)			Percent Change Through Retention Basin	
Parameter	Dissolved (µg/L)	Total (µg/L)	Percent Dissolved	Dissolved (µg/L)	Total (µg/L)	Percent Dissolved	Dissolved	Total
Cd	1.1	1.9	58	0.8	1.0	80	-27	-47
Zn	50	347	14	5.8	6.4	91	-88	-98
Cu	32	60	53	14	16	88	-56	-73
Pb	43	723	6	16	22	73	-63	-97
Ni	3.2	28	11	1.8	2.3	78	-44	-92
Cr	3.3	10	33	2.3	3.4	68	-30	-66
Fe	48	1176	4	20	61	33	-58	-95

Cu, 94 percent of the total Pb, 89 percent of the total Ni, 67 percent of the total Cr, and 96 percent of the total Fe, as shown in Table 4. All the soluble and particulate fractions in the pondwater were lower than those detected in the incoming stormwater. The reduction in concentrations varied between 27 and 88 percent in the dissolved fraction and between 47 and 98 percent in the total metal concentration.

The results previously presented indicate that the fate of a large portion of both the suspended and dissolved fractions of stormwater-associated heavy metals is the ultimate deposition of a wide variety of mechanisms into the bottom sediments of the receiving water body. After several years of this continual deposition, a large accumulation of heavy metals may develop in the sediments. This concentrated layer of heavy metals may present a potential pollution hazard if leaching occurs. To investigate the potential movement of sediment-deposited heavy metals, the vertical distribution of heavy metals in 43 sediment cores collected from the Maitland ponds was examined and the average metal concentrations in sediment layers of 0 to 0.8, 0.8 to 2.8, 2.8 to 4.8, and 4.8 to 6.8 cm were calculated.

The heavy-metal content in the 4.8 to 6.8-cm layer was similar to heavy-metal concentrations measured in nearby soils that were unaffected by stormwater runoff. Therefore, these concentrations were considered equal to background values and subtracted from each of the others. The vertical distributions of Zn, Pb, Cr, Ni, Cu, and Fe in sediment cores that were collected in the West pond are shown in Figure 3. This figure shows that the metal concentrations decreased in an exponential fashion with a correlation coefficient of 0.99 or better. Accumulated heavy metals were quickly attenuated during movement through sediment material. Attenuation of the metals was found to occur in the top 5.0 to 6.8 cm with normal background concentrations below that depth.

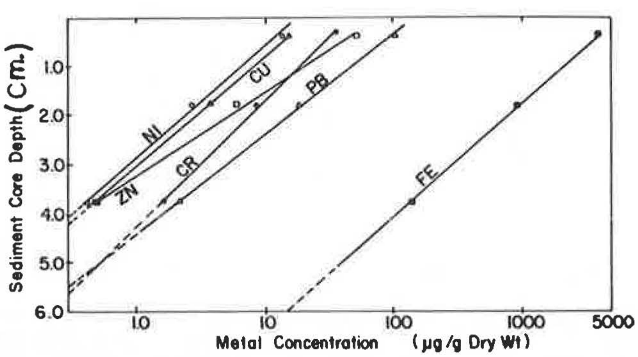


FIGURE 3 Transports of heavy metals through bottom sediments of retention/detention pond at Maitland Interchange.

It can be concluded, therefore, that heavy metals, after reaching the sediments, are transformed into stable associations that remain near the sediment surface and decline rapidly in concentrations with increasing depth.

Most previous studies that deal with particulate metal fractions have measured total metal concentrations, whereas relatively few attempts have been made to evaluate the speciation of particulate metals. The use of total metal concentration as a basis for evaluating particulate matter or sediments implies that all forms of a given metal are equal in terms of their toxicity, biological and physiological availability, mobility, and origin. Obviously, this assumption is not valid.

Theoretically, it is chemically possible to partition solid material into specific metal fractions by using appropriate procedures and an improved sequential extraction procedure for the speciation of particulate trace metals that had been reported (9). In this procedure, six fractions can be identified that eliminate many of the problems previously reported in single extraction procedures.

The data indicate that very little or none of the trace metals in the sediments is soluble in distilled deionized water. Pb appears to exist primarily in exchangeable form and associated with Fe-Mn oxides. Cu, Fe, Cr, Ni, and Zn are mainly bound to Fe-Mn oxides and with organic matter in the sediments. Also, most of the Cd in sediments appears to be exchangeable. Therefore, the potential of trace metal release by natural water is very limited or unlikely if aerobic conditions are maintained. Under aerobic conditions, hydrous Fe-Mn oxides act as a sink for trace metals that result from specific adsorption and coprecipitation.

PREDICTION OF METAL SPECIATION

The speciation of trace elements in natural waters is important in the assessment of the potential for biological uptake. Most of these analytical techniques measure gross parameters such as total dissolved Pb or Cu but give no clue as to the actual form of their existence in the environment. One of the methods used to gain insight into this area is computer modelling. One widely used computerized chemical model for trace and major element speciation and mineral equilibria of natural waters is WATEQ2 (5). This computer model can be used to predict the average dissolved metal species present in water samples of known chemical composition. The predicted average dissolved metal species in water samples from the study area are shown in Table 5.

The predicted speciation shows that Zn and Cd exist mainly as free ions below pH 8 and are controlled by carbonates at higher pH values. Pb exists as free ions, bicarbonate, and carbonate forms below

TABLE 5 Predicted Average Dissolved Metal Species in Water Samples from Study Areas Using WATEQ2 Model

		Average Concentration (μg/L)				
Metal	Major Species	Rainwater	Highway Runoff	Pond Water	Bridge Runoff	Shingle Creek Water
Zn	Zn^{+2}	8.1	19	9.3	11.3	12.8
	$ZnHCO_3^+$	0.1	2.3	0.9	1.7	1.2
	$ZNCO_3$	-	1.2	0.4	1.6	0.5
	Total	8.2	23.0	11.0	15.0	15.0
Cd	Cd^{+2}	2.46	1.28	0.82	2.15	1.44
	$CdHCO_3^+$	0.04	0.16	0.08	0.30	0.13
	$CdCO_3$	-	0.10	0.05	0.40	0.06
	Cd-Fulvate	-	0.02	0.01	0.06	0.06
	Total	2.50	1.60	1.00	3.00	1.80
Pb	Pb^{+2}	36.6	5.7	3.8	1.8	3.7
	$PbHCO_3^+$	0.7	4.3	2.4	1.8	2.2
	$PbCO_3$	0.2	30.3	13.0	23.8	12.1
	Total	38.0	41.0	20.0	28.0	19.0
Cu	Cu^{+2}	34.4	2.4	1.8	0.8	3.4
	Cu-Fulvate	30.0	16.1	9.8	10.8	7.7
	$Cu(OH)_2$	-	2.4	1.2	3.2	0.2
	$CuCO_3$	-	3.9	1.9	3.3	0.3
	Total	66.0	27.0	16.0	19.0	9.0
Ni	Ni^{+2}	0.99	0.87	0.76	0.45	1.17
	$NiCO_3$	-	2.00	1.13	2.46	1.68
	Total	1.00	3.00	2.00	3.00	3.00

pH 6.5 and is controlled mainly by the carbonate concentration above pH 7. Ni exists mainly as the free metal ion below pH 6, is divided between the free ion form and $NiCO_3^o$ between pH 6 and 7.5, and is controlled solely by carbonate concentration above pH 7.5. Finally, Cu exists mainly as Cu-fulvate between pH 5 and 7, is divided between Cu-fulvate and $Cu(OH)_2^o$ between pH 7 and 8, and is controlled by $Cu(OH)_2^o$ exclusively above pH 8.

The predicted species show specific compounds that are based on thermodynamic data, and the measured species using ASV are grouped in classes on the basis of behavioral characteristics. However, for the purpose of comparison, it may be assumed that free metal ion is represented by the measured class of soluble labile and that fulvate-humate compounds represent the organic fractions measured by ASV. The predicted and measured speciation indicated that Zn and Cd existed mainly as free metal ions in natural waters. Cu is strongly influenced by organic matter present and exists mainly as organic complexes. However, organic Pb complexes are measured by ASV and are not predicted by WATEQ2 as a result of the lack of sufficient thermodynamic data for organic lead compounds.

Of course, it is very difficult to find in the literature the thermodynamic data on metal complexes that are associated with fulvic and humic substances in the natural environment. As more information becomes available, the computer program can be modified and improved. The organic Pb complexes varied between 15 and 44 percent of the metal in solution measured by ASV. The labile Pb fraction appears to include ionic forms as well as organic and inorganic complexes. Also, most of the labile fraction of Cu in the aquatic environment may be associated with organic complexes.

The presence of organic substances in natural waters and the role of the sediments in the removal and retention of trace metals that are released to receiving streams tend to detoxify the metals associated with highway runoff. Most of the metals are retained by the bottom sediments on a permanent basis if aerobic conditions and high redox-potential (Eh) values are maintained. The ability of natural waters to detoxify trace metals was demonstrated by the higher lethal concentrations for 50 percent kill (LC-50) for mosquito fish in retention/detention pondwater as compared with the LC-50 values in deionized tap water (10).

CONCLUSIONS

Trace metals in aqueous systems are partitioned between the overlying water column and the accumulated sediments beneath it. Heavy metals that are bound within the crystal lattice of clay particles are considered unavailable, whereas materials dissolved in interstitial or surface adsorbed ions that may be easily displaced by ion exchange are considered available. Between those two extremes, heavy metals may be present in chemical forms that are potentially available to organisms. There are several similar classification schemes for dissolved heavy metals in natural water systems. These schemes may assist in the identification of the most toxic species in solution that are available to biota. It is realized that biotoxicity is dependent on the available species and not on the total metal concentration. During this study, the following conclusions were reached:

1. Retention/detention ponds similar to the Maitland ponds are effective in nutrient and heavy-metal removal from highway runoff. The bottom sediments concentrate the heavy metals and nutrients that are discharged into the pond.
2. The average dissolved Cd, Zn, Cu, Pb, Ni, Cr, and Fe in the Maitland ponds are 73, 12, 44, 37, 56, 70, and 42 percent of those detected in the incoming highway runoff, respectively. Similarly, the total Cd, Zn, Cu, Pb, Ni, Cr, and Fe concentrations in the Maitland ponds averaged 53, 2, 27, 3, 8, 34, and 5 percent of those measured in the incoming highway runoff. Most of the metals in highway runoff appear to be in particulate form.
3. Bottom sediments in the Maitland ponds have heavy-metal concentrations in the top 5 to 6.8 cm. Accumulated heavy metals are attenuated very quickly during movement through sediment material. Most of the metals are mainly bound to Fe-Mn oxides and organic matter. A fraction of Pb and Cd appears to be exchangeable. Therefore, the potential of trace metal release to solution by natural water is very limited or unlikely if aerobic conditions are maintained.
4. More than 70 percent of the soluble Cd and Zn in rainfall, highway runoff, Maitland pondwater, and Shingle Creek water exists in ionic form (M^{+2}). Most of the Pb exists as $PbCO_3^o$, and a significant fraction of the Cu is associated with organic complexes if humic substances are present.
5. Existing standards for trace metals do not specify chemical speciation of those metals or their bioavailability. The detoxifying effect of some water quality parameters on trace metals should be reflected in the standards.
6. Determination of existing chemical speciation by computerized models can be a useful tool for planning purposes in the prediction of potential environmental impacts.

ACKNOWLEDGMENTS

The authors wish to acknowledge with gratitude the financial support and technical assistance given by FDOT and FHWA personnel. In particular, the interest and assistance of Gary Evink and Larry Barfield from FDOT and Byron Lord from FHWA are very much appreciated.

REFERENCES

1. Y.A. Yousef, M.P. Wanielista, H.H. Harper, and J.E. Christopher. Management of Drainage Systems from Highway Bridges for Pollution Control. In Transportation Research Record 896, TRB, National Research Council, Washington, D.C., 1982, pp. 51-55.
2. Y.A. Yousef, M.P. Wanielista, T. Hvitved-Jacobsen, and H.H. Harper. Fate of Heavy Metals in Stormwater Runoff from Highway Bridges. In the Science of the Total Environment, Vol. 33, Elsevier Science Publishers, B.V., Amsterdam, Netherlands, 1984.
3. G.E. Bately and T.M. Florence. A Novel Scheme for the Classification of Heavy Metal Species in Natural Waters. Analytical Letters, Vol. 9, 1976, p. 379.
4. W.G. Wilber and J.V. Hunter. Aquatic Transport of Heavy Metals in the Urban Environment. Water Resources Bull., Vol. 13, No. 4, 1977, pp. 721-734.
5. J.W. Ball, D. Nordstrom, and E.A. Jenne. Additional and Revised Thermochemical Data and Computer Code for WATEQ2--A Computerized Model for Trace and Major Element Speciation and Mineral Equilibria of Natural Waters. WRI 78-116. U.S. Geological Survey, Menlo Park, Calif., 1980.
6. Y.A. Yousef, H.H. Harper, Lee Wiseman, and Mike Bateman. Consequential Species of Heavy Metals in Highway Runoff. Report FL-ER-29-85. Florida Department of Transportation, Tallahassee, 1985.
7. Methods for Chemical Analysis of Water and Wastes. Report EPA-60014-79-020. U.S. Environmental Protection Agency, March 1979.
8. T. Hvitved-Jacobsen, Y.A. Yousef, M.P. Wanielista, and D.B. Pearce. Fate of Phosphorus and Nitrogen in Ponds Receiving Highway Runoff. In the Science of the Total Environment, Vol. 33, Elsevier Science Publishers, B.V., Amsterdam, Netherlands, 1984.
9. A. Tessler, P.G.C. Campbell, and M. Bisson. Sequential Extraction Procedure for the Speciation of Particulate Trace Metals. Analytical Chemistry, Vol. 51, No. 7, 1979.
10. J.M. Bateman. Bio-toxicity of Highway Runoff Metals to Gambusia Affinis. M.S. thesis. Department of Civil Engineering and Environmental Sciences, University of Central Florida, Orlando, 1984.

Removal of Highway Contaminants by Roadside Swales

Y. A. YOUSEF, M. P. WANIELISTA, and H. H. HARPER

ABSTRACT

Removal of highway contaminants by roadside swales was investigated at the Maitland and EPCOT Interchanges with Interstate 4 (I-4) in Orange County, Florida. Runoff samples from highway and grassy swale areas at Maitland Interchange were collected for 8 months for comparison of highway runoff with runoff that passed through a grassy swale. Also, a controlled water flow from adjacent detention/retention ponds was dosed with nitrogen, phosphorus, and heavy metals to produce concentrations typical of highway runoff. The mixture was allowed to flow for a period of 3.0 to 5.5 hr over selected areas of adjacent roadside swales. Periodic grab samples were collected from several locations along the swale throughout the flow period and were analyzed to determine concentration and mass removal rates for various pollutants under several values of flow rates and experimental conditions. Hydraulic, hydrologic, and water quality parameters were evaluated. Removal efficiencies for dissolved heavy metals appeared to be higher than for nitrogen and phosphorus. Pollutants may be retained in swale areas by sorption, precipitation, coprecipitation and biological uptake processes; however, occasional increases in concentrations of highway contaminants were observed at intermediate stations during the swale experiments. Mass removal of heavy metals, nitrogen, and phosphorus was directly related to infiltration losses and on-site storage.

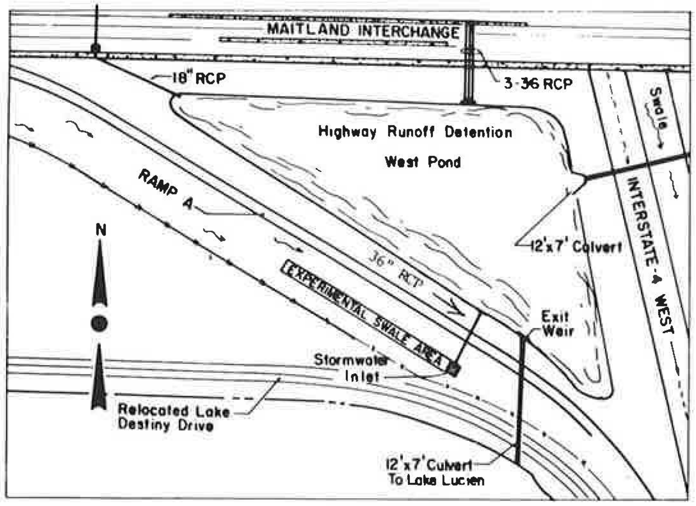


FIGURE 1 Location of experimental swale along Ramp A at Maitland Interchange and I-4.

Highways are routinely drained through broad, shallow, grassed channels, often termed swales, which attenuate runoff hydrographs and control roadside erosion. The hydraulic efficiencies of swales are based on the ability to infiltrate and percolate stormwater. However, their pollutant removal efficiencies, on the basis of quality considerations, have not been determined and little documentation on water quality effects is available. To the authors' knowledge, the only previous study on this subject was performed by Wang et al. (1). The Wang study was limited in scope and was designed to measure heavy metal concentrations in stormwater runoff that drained through a paved channel, a mud-bottomed ditch, and channels vegetated with grasses. No general conclusions were reached; however, it was reported that a 60-m-long grass channel of small slope reduced highway-runoff dissolved lead (Pb) concentrations by 80 percent or more during flow through the channel. Removal efficiencies averaged 60 percent for dissolved copper (Cu) and 70 percent for dissolved zinc (Zn) in the same channel. Samples from bare earthen channels of 15-m length and paved channels did not indicate significant reductions in heavy metal concentrations; however, no mechanisms were suggested for the observed results.

During this study, analyses of water quality of highway runoff and flow from grassy swales were conducted and the results compared. Also, a continuous flow of simulated highway runoff was pumped over the experimental area of a swale at the Maitland Interchange and I-4 and at the EPCOT Interchange and I-4 sites near Orlando in Orange County, Florida (Figures 1 and 2). Controlled experiments were designed to investigate changes in pollutant concentrations and mass balances for highway runoff that flows along swale areas. An attempt is made to answer some of the following questions:

1. Are swales efficient in phosphorus and nitrogen removal from stormwater runoff? If they are, to what extent and for how long do they retain those nutrients?

2. Are swales efficient in heavy metal removal from highway runoff? If they are, to what extent and for how long do they retain those metals? Also, are there different affinities for different metals and under what conditions are these metals released?

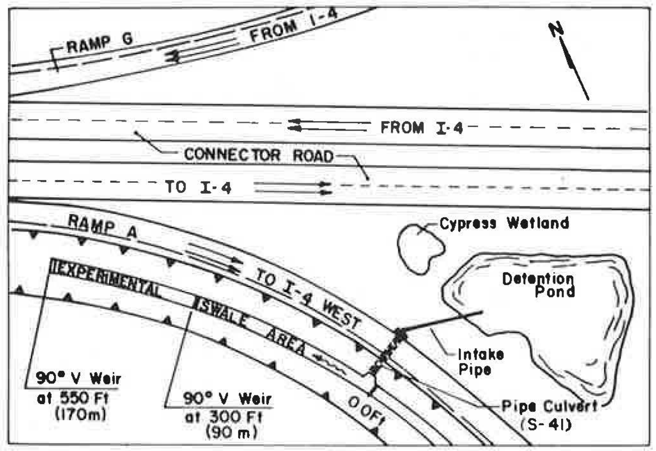


FIGURE 2 Location of Experimental Swale along Ramp A at I-4 and EPCOT Interchange.

3. Is it possible to develop design considerations on the basis of pollutant removal efficiencies?

FIELD EXPERIMENTATION

Concentrated solutions of heavy metals, such as Pb, chromium (Cr), cadmium (Cd), Nickel (Ni), Cu, Zn and iron (Fe) as well as nutrients such as phosphorous (P) and nitrogen (N), were dissolved in a 120-L polyethylene container. The chemical spikes were fed at a fairly constant rate into the flowing water before entering the swale area by using constant head gravity flow or a peristaltic dosing pump with controlled flow discharge. At the Maitland site, the chemical solution was fed by gravity near the inlet side of the submersible pump that was placed in the stormwater inlet (see Figure 1) and that was used for water intake. At the EPCOT site, the chemical solution was dosed by a peristaltic pump at the inlet of the 18-in. pipe culvert (S-41) shown in Figure 2 that crosses through ramp A to the start of the swale area. The spiked water was allowed to flow by gravity over the swale areas at selected discharge rates that were controlled by a gate valve attached to the PVC pipe near the suction side of the submersible pump.

TABLE 1 Hydraulic Characteristics of Swale Experiments

Experimental Swale					Flow Characteristics					
								Q (m^3/min)		
No.	Location	Date	Length (m)	Duration (hr)	Cross-Section Area (m^2)	Top Water Width (m)	Hydraulic Depth (m)	In	Out	Avg Calculated Velocity (m/min)
1	Maitland	1/24/83	53	3.00	0.063	1.67	0.038	0.227	0.098	2.58
2	Maitland	2/07/83	53	4.00	0.045	1.35	0.033	0.086	0.038	1.37
3	Maitland	2/21/83	49	5.50	0.014	0.85	0.017	0.026	0.000	0.90
4	EPCOT	3/23/83	90	4.18	0.058	1.46	0.040	0.189	0.131	2.76
			80		0.060	1.49	0.040	0.131	0.118	2.08
5	EPCOT	5/16/83	90	4.00	0.056	1.37	0.041	0.189	0.145	2.98
			80		0.071	1.83	0.039	0.145	0.131	1.94
6	Maitland	5/31/83	53	4.00	0.056	1.79	0.031	0.145	0.118	2.35

The swale areas at the Maitland site were covered with predominantly Bahia grass that was approximately 2 to 4 in. high. A total of five flow-through experiments were conducted from January to May 1983. During January and February, the grass cover was dry and dormant. In late March, however, the grass cover was green and growing. The swale areas at EPCOT were newly constructed and consisted of bare soil with a 20 percent Bahia grass cover during the first experiment on March 23, 1983, and an 80 percent weed-and-Bahia grass cover during the second experiment on May 16, 1983.

Grab samples of water flowing over the swale area were periodically collected at several locations and transported to the Environmental Engineering and Sciences Laboratory at the University of Central Florida in Orlando for analysis. The analyses were conducted within the time frame recommended by the U.S. Environmental Protection Agency (3).

HYDROLOGIC AND HYDRAULIC PARAMETERS

The pumped inflow rates varied between 0.026 and 0.227 m^3/min (7 to 60 gal/min) for the Maitland area and averaged 0.189 m^3/min (50 gal/min) for the EPCOT area. The hydraulic characteristics of the swale experiments are listed in Table 1. The hydraulic water depth, which is defined as the cross sectional area divided by the top width of flow did not exceed 0.0041 m (1.6 in.). The calculated water velocity varied from 0.90 to 2.98 m/min (0.05 to 0.16 ft/sec) during the swale experiments under steady state conditions of flow.

After cessation of pumping, the flow through the exit weir was monitored until flow no longer occurred to produce the shape of the fall of the hydrograph. Flow hydrographs were then developed at each experiment. During the February 21, 1983 experiment at the Maitland site, the water did not reach the end of the swale and it was totally retained on the site. (Water is lost by infiltration, seepage, evaporation, transpiration and on-site storage.)

The hydrographs clearly reflect the water retention and the excess runoff from swale areas under various inflow rates. Hydrograph characteristics of the swale experiments are given in Table 2. The average loading rates varied from 0.036 to 0.154 $m^3/(m^2 \cdot hr)$ (1.42 to 6.06 in./hr) on the Maitland swale area. The rates resulted in excess runoff averaging 0.0 to 0.068 $m^3/(m^2 \cdot hr)$ (0 to 2.7 in./hr). The EPCOT site loading rates averaged 0.053 to 0.105 $m^3/(m^2 \cdot hr)$ (2.08 to 4.13 in./hr) and the excess runoff averaged 0.039 to 0.071 $m^3/(m^2 \cdot hr)$ (1.52 to 2.8 in./hr). The flow rates were calculated from the area under the hydrograph divided by the submerged area of the swale and the duration time of the flow. Under the experimental conditions, there was no excess runoff for flow less than 1.42 in./hr. Excess runoff reached more than 90 percent of average input flow at the EPCOT site when the soil was saturated with moisture.

TABLE 2 Hydrograph Characteristics of Swale Experiments

	Mass Flow				
No.	In (m^3)	Out (m^3)	Loading Rates [$m^3/(m^2 \cdot hr)$]	Infiltration [$m^3/(m^2 \cdot hr)$]	Excess Runoff [$m^3/(m^2 \cdot hr)$]
1	40.9	17.6	0.154	0.088	0.066
2	20.7	8.3	0.072	0.043	0.029
3	8.14	0	0.036	0.036	0
4	57.7	39.2	0.105	0.034	0.071
	39.2	35.8	0.079	0.007	0.072
5	46.3	30.8	0.094	0.032	0.062
	30.8	23.0	0.053	0.014	0.039
6	35.1	26	0.092	0.024	0.068

QUALITATIVE ANALYSIS

This study included the collection of water quality parameters from six different stations that surround the Maitland Interchange for approximately 8 months during 1982-1983 (3). Sampling stations were divided between stations that collect direct highway runoff and stations that collect flow from grassy swales. Comparison of average concentrations in water samples showed that both total and dissolved forms of every metal analyzed were lower in swale flow than highway flow as shown in Table 3. On the contrary, water samples from swale flow contained higher average concentrations of the nutrients, P and N than highway runoff.

TABLE 3 Comparison of Average Concentrations in Highway Runoff and Swale Flow During 1982-1983 at Maitland Interchange

	Avg. Concentration (μg/L)					
	Highway Runoff (N=48)		Swale Flow (N=25)		Percent Change	
Pollutant	Total	Dissolved	Total	Dissolved	Total	Dissolved
Zn	225	69	25	16	-90	-82
Pb	417	36	36	18	-91	-50
Cu	44	27	26	22	-41	-19
Fe	830	52	240	29	-71	-44
Cr	6.2	3.2	4.6	2.8	-44	-13
Cd	1.4	1.1	1.0	0.9	-29	-18
Ni	21	3.4	2.4	1.8	-86	-47
OP-P	136	84	239	195	+76	+132
TP-P	280	NA	351	NA	+25	NA
NH_4-N	NA	118	NA	172	+47	NA
(NO_2+NO_3)-N	NA	280	NA	261	-7	NA
Organic N	NA	1732	NA	1784	+2	NA
Total N	NA	2130	NA	2217	+4	NA

Note: NA = Not available.

Zn concentrations were decreased to the greatest degree through swale areas with average reductions of 82 and 90 percent in dissolved and total species, respectively. Concentrations of Pb decreased 91 and 50 percent for total and dissolved species, respectively. Cu, Cd, and Cr concentrations decreased to the least degree through swale areas with an average reduction of 29 to 44 percent in total metal and 13 to 19 percent for the dissolved metal. It is assumed that the particulate metal fractions are removed through the swales at a higher percentage than the dissolved fraction as a result of filtration of particulate matter during transport through grassy covers.

Similar results were obtained during the simulated continuous flow swale experiments performed at the Maitland and EPCOT sites. Dissolved pollutants in flowing water at various locations along experimental sites of roadside swales are presented in Table 4. The Maitland site showed the greatest reduction in dissolved Zn and Fe with averages of 86 and 69 percent removal, respectively. Removals of dissolved Pb, Cu, and Cr were not significant. Under controlled conditions, however, reductions in nutrient (N and P) concentration were indicated that averaged 24, 25, 31, 13, and 6 percent for OP-P, TP-P, NH_4-N, $(NO_2 + NO_3)$-N and TN, respectively. Organic nitrogen increased in concentration by 13 percent after flowing 53 m along the swale area.

TABLE 4 Average Concentrations of Dissolved Pollutants Flowing over Roadside Swales

	Avg Concentration (μg/L)						
	Maitland Site			EPCOT Site			
Pollutant	0.0 m	23 m	53 m	0.0 m	30 m	90 m	170 m
Zn	22	9	3	140	103	77	53
Pb	9	5	9	67	43	41	29
Cu	6	6	5	26	30	29	24
Fe	260	102	81	290	290	261	316
Cr	9	6	8	10	9	10	10
Cd	–	–	–	7	6	5	4
Ni	–	–	–	70	59	47	34
OP-P	368	290	279	580	546	514	530
TP-P	415	367	310	599	586	558	580
NH_4-N	1015	870	699	293	321	297	299
(NO_2+NO_3)-N	192	188	167	147	151	147	163
Organic N	842	1337	951	1833	1994	1683	1973
Total N	2049	2395	1817	2273	2456	2127	2435

Removal rates of 62, 57, 43, and 51 percent for Zn, Pb, Cd, and Ni, respectively, were observed at the EPCOT site. All other measured parameters decreased slightly or increased after flowing 170 m. This site was a newly constructed swale with very little vegetation, a high water table elevation, and was nearly saturated with water during the swale experiments.

TOTAL MASS REMOVALS

Removal of pollutants in terms of a reduction in concentration is a useful approach because many water quality regulations are based on allowable concentrations that enter waterways rather than on a more difficult total mass approach. These regulations were presumably developed in this fashion to simplify enforcement. A small discharge that violates certain parameters of the regulation, however, may be far less damaging on a long-term basis than a continuous input that meets the regulations.

If the removal of pollutants in roadside swales is considered on a total mass basis, the removal efficiencies will increase considerably as shown in Tables 5 and 6. Not only must the removal as a result of a change in concentration through the swale be considered, but also must the removal by infiltration into the ground be considered. This approach can be carried out to an extreme in a case such as the third Maitland experiment that showed 100 percent mass removal rates for all pollutants tested since the swale flow did not reach the outfall. It is interesting to note that some parameters such as Pb in the first Maitland experiment showed a 25 percent increase in concentration during passage through a swale area and was reduced in total mass by 46 percent.

Mass removal rates in terms of $mg/(m^2 \cdot hr)$ for each of the Maitland and EPCOT experiments are also listed in Tables 5 and 6. These rates are calculated by dividing the total mass removed during travel through the swale area by the wetted area and experiment duration. It is important to note that these rates are highly site-specific. Mass is retained in the swale area by infiltration, seepage, transpiration, and soil-grass-water interactions. As runoff water is stored and retained in the swale area, the percent mass removal efficiency is increased. Water retention is a function of infiltration rate that may be related to surface water velocity or the residence time through the swale. Of course, infiltration is a function of many variables such as the antecedent dry period, soil porosity, and cover crop. However, it appears that the lower the surface velocity, the greater the infiltration rate along the swale, if other factors are assumed to be constant. Good correlations were found between mass removal rates of phosphorous, nitrogen species, and infiltration rates (3). Also, positive correlations between mass removal rates and total mass input were observed for most of the metals that were tested. This relationship is intuitively correct because the removal model is influenced by adsorption onto soil particles. This adsorption process should be expected to increase as the driving force in the form of total mass input increases.

DISCUSSION

The experimental swales were built with side slopes that were more than 6 horizontal to 1 vertical and longitudinal slopes of approximately 0.6 percent for the Maitland site and 0.1 percent for the EPCOT site. The field data included excess runoff, swale slope, average time of concentration, and length of travel; therefore, a relatively accurate estimate of the channel roughness measured by Manning's coefficient n could be calculated (4). The values for Manning coefficients varied from 0.055 to 0.096 for Maitland swale and from 0.035 to 0.059 for the EPCOT swale. The overall average value for n was 0.06. In the Maitland experiment conducted on February 7, 1983, the roughness coefficient was as high as 0.096. If this value was discarded when an average value for roughness was calculated, the mean coefficient would be 0.05. This value may help other designers of swale systems.

Also, when the rational formula was used, peak discharge for the shorter swale (53 m) was accurately predicted. However, for the larger swale (170 m), the calculated value was not as accurate when compared with the measured value. The average drainage area may be the variable that was difficult to measure accurately (4). The rational formula appears to be applicable for calculating peak discharges through swale areas.

TABLE 5 Removal of Dissolved Pollutants Flowing Over Roadside Swales at Maitland Interchange and I-4

		Mass Balance				Avg Concentration		
Date	Pollutant	In (g)	Out (g)	Percent Change	Removal [mg/(m^2 · hr)]	In (μg/L)	Out (μg/L)	Percent Change
1/24/83	Zn	0.900	0.09	-90	3.1	22	5	-77
	Pb	0.49	0.26	-46	2.8	12	15	+25
	Cu	0.33	0.12	-62	0.8	8	7	-13
	Fe	1.43	0.30	-79	4.3	35	17	-51
	Cr	0.49	0.21	-57	3.4	12	12	0
	TP	25.60	9.47	-63	61	625	538	-14
	OP	23.80	8.98	-62	56	582	510	-12
	Ing-N	45.03	17.49	-58	104	1101	994	- 9.7
	Org-N	8.27	3.57	-64	18	202	203	0
	T-N	53.21	2.09	-61	122	1301	1190	- 8.5
2/7/83	Zn	0.54	0.02	-97	1.8	26	2	-92
	Pb	0.17	0.05	-70	0.4	8	6	-25
	Cu	0.15	0.03	-83	0.4	7	3	-57
	Fe	9.71	1.19	-88	30	469	143	-70
	Cr	0.12	0.03	-73	0.3	6	4	-33
	TP	12.15	2.55	-79	33	587	307	-48
	OP	9.89	2.10	-79	27	478	253	-47
	Ing-N	24.90	6.77	-73	62	1203	816	-32
	Org-N	6.56	2.65	-60	13	317	319	0
	T-N	31.46	9.41	-70	75	1520	1134	-25
2/2/83	Zn	0.15	0	-100	0.66	18	NA	NA
	Pb	0.05	0	-100	0.22	6	NA	NA
	Cu	0.02	0	-100	0.07	2	NA	NA
	Fe	2.56	0	-100	11.2	314	NA	NA
	TP	1.69	0	-100	7.4	207	NA	NA
	OP	1.64	0	-100	7.2	201	NA	NA
	Ing-N	7.12	0	-100	32	875	NA	NA
	Org-N	1.83	0	-100	9	225	NA	NA
	T-N	8.95	0	-100	41	1100	NA	NA
5/31/83	TP	8.39	5.77	-31	6.9	234	221	- 6
	OP	7.41	5.46	-26	5.1	211	210	0
	Ing-N	57.8	37.1	-36	52	1646	1427	-13
	Org-N	92.0	71.6	-22	51	2621	2755	+ 5
	T-N	149.8	108.7	-27	103	4267	4182	- 2

Note: NA = not available.

TABLE 6 Removal of Dissolved Pollutants Flowing Over Roadside Swales at EPCOT Interchange and I-4

		Mass Balance				Avg Concentration		
Date	Pollutant	In (g)	Out (g)	Percent Change	Removal [mg/(m^2 · hr)]	In (μg/L)	Out (μg/L)	Percent Change
3/23/83	Zn	14.77	3.54	-76	10.7	256	99	-61
	Pb	5.25	1.15	-78	3.9	91	32	-65
	Cu	1.27	0.68	-46	0.56	22	19	-14
	Fe	28.22	18.54	-34	9.2	489	518	+6
	Cr	0.64	0.29	-55	0.33	11	8	-27
	Cd	0.64	0.25	-61	0.37	11	7	-36
	Ni	6.46	1.86	-71	4.4	112	52	-54
	TP	62.2	35.2	-43	25.8	1078	983	-9
	OP	62.1	34.4	-45	26.4	1077	960	-11
	Ing-N	15.5	10.7	-31	4.6	268	300	+12
	Org-N	100	59.4	-41	38.7	1733	1658	-4
	T-N	115	70.1	-39	42.8	2001	1959	-2
5/16/83	Zn	1.07	0.18	-83	0.82	23	8	-65
	Pb	2.04	0.60	-71	1.32	44	26	-41
	Cu	1.34	0.64	-52	0.64	29	28	-3
	Fe	4.21	2.62	-38	1.46	91	114	+25
	Cr	0.37	0.21	-43	0.15	8	9	+13
	Cd	0.09	0.02	-78	0.06	2	1	-50
	Ni	1.30	0.37	-72	0.85	28	17	-43
	TP	5.56	4.05	-27	1.39	120	178	+48
	OP	3.84	2.30	-40	1.42	83	100	+20
	Ing-N	28.3	14.3	-49	12.9	613	624	+2
	Org-N	89.5	52.6	-41	33.9	1932	2288	+18
	T-N	117.8	67	-43	46.7	2545	2912	+14

Data collected over an 8-month period from grab samples of both highway and swale runoff indicate lower removal efficiencies than were obtained in the experimental controlled flow situations. It is possible that certain metals may change forms between storm events and become soluble. This is particularly likely in elements that have a change in species from a charged free ion to a neutral ion in the pH range of from 6.0 to 7.5.

From the results obtained in these swale experiments, it appears that the chemistry of heavy metals in natural waters is a fairly complex and site-specific phenomenon. In the studies conducted at Maitland in which only inorganic species were assumed to be present, the solubility and removal efficiencies that were obtained for dissolved species appeared to be related to the dominant inorganic complex present. Those metal species that were present as a charged ion, such as Zn and Fe, were removed to a significant degree. Those that were complexed with inorganic species and that carried either a diffuse or zero charge were not removed.

The importance of organic complexing in regulating solubility was demonstrated through the EPCOT experiments. Of the metal ions present, Cu, and Fe are known to form significant metal-organic complexes and, as a result, no removal was found to occur. Other metals that formed no important organic complexes were regulated by their inorganic species.

Ionic nitrogen species (NH_4^+, NO_2^-, NO_3^-) and phosphorous species ($H_2PO_4^-$, HPO_4^{-2}, PO_4^{-3}) may similarly be retained on the swale site by sorption, precipitation, coprecipitation and biological uptake processes. By these processes, reductions of the nutrient concentrations in highway runoff that flows over swales can be made. In addition, a thin grass cover (20 percent or less) seems to be more efficient in decreasing contaminants than a thick grass cover (80 percent or more). It is believed that a thick grass cover may affect available soil sorption sites and increase organic debris (grass clippings, mower debris, litter). The organic debris is then subjected to decay processes and relocation. This was evident from the decline in the removal efficiency of soluble NO_3^- and NH_4^+ forms of N and organic-N in thick grassy swales that was observed on May 16, 1983 at the EPCOT site and on May 31, 1983 at the Maitland site. Also, the decrease in the removal of organic-N concentration may be attributed to an increase in organic deposition in the swale as a result of organic debris that exists during periods of rapid grass growth.

Occasional increases in highway contaminants were observed at intermediate stations during swale experiments, particularly at stations located close to the inflow point. This appears possible because of the initial flow effects on resuspension and resolubilization of loosely bound contaminants. The swale experiments showed better removal efficiencies at slow rather than high flow rates. The removal of nitrogen in swales on a concentration basis (measured in this study as micrograms per liter) was found to be inversely related to the velocity of the runoff through the swale (i.e., directly related to the residence time of the runoff in the swale). There seems to be very little removal of N concentrations when the excess runoff is above 3 in./hr. It is therefore apparent that if swales are designed to produce low inflow rates and velocities, some N concentration removal could be expected, with the amount of removal being a function of site conditions, such as swale cover and soil characteristics.

The removal of heavy metals, N, and P species on a mass basis, is directly related to infiltration losses through swales; therefore, retention of as much water as possible on the swale area will reduce the highway contamination loadings to adjacent receiving waters.

CONCLUSIONS

The following conclusions were reached:

1. Minimum observed infiltration rates were 0.5 and 1.4 in./hr and maximum rates were 1.3 and 3.4 in./hr for swales studied at EPCOT and Maitland Interchanges, respectively. These rates are much lower than rates measured by the double-ring infiltrometer technique that shows values three to four times higher.
2. The measured runoff coefficients depend on the degree of soil saturation and the antecedent dry period. They varied between 0.41 and 0.91 during this investigation. Also, the calculated Manning's friction coefficient n for flow through the swale generally varied between 0.035 and 0.059 with an average value of 0.053 for most of the cases.
3. Swales built on high ground with good drainage and high infiltration rates showed better removal efficiencies for highway contaminants. Results from EPCOT suggest that removal of heavy metals decreases significantly in swale areas that are low and constantly wet.
4. Swales appear to be more effective in reducing concentrations of metals that N and P in a flow-through situation. These efficiencies are governed by the predominant ionic species and complexes. Charged species are retained by sorption processes. Swales filter out particulate heavy metal and incorporate them into the soil. Heavy metals in highway runoff with large particulate fractions show higher removal efficiencies.
5. Removal of heavy metals may be caused by precipitation and sorption processes; therefore, charged ions and complexes may be removed more efficiently than stable complexes and noncharged particles.

RECOMMENDATIONS

The efficiency of swale for removal of pollutants can be increased by increasing contact time and infiltration rates; therefore, the following recommendations are made:

1. Reduce longitudinal slopes of future roadside swales as much as possible;
2. Increase contact surface within the swale area by increasing the wetted perimeter-to-cross section area ratio by using relatively flat side slopes whenever possible;
3. Whenever possible, avoid building swales in areas where portions remain wet most of the time as a result of low ground and high groundwater tables;
4. Construct earthen cross barriers (swale blocks) at selected length intervals along the swale to retain additional water, so that the maximization of on-site retention by storage of runoff water in swales built on upland areas may be achieved; and
5. Plant a cover crop for erosion control and follow effective maintenance procedures; removal of grass clippings, loose debris, and litter is desirable if practical; also, consider slow growing grass species with low maintenance requirements.

ACKNOWLEDGMENTS

The authors wish to acknowledge with gratitude the financial support and technical assistance given by FDOT and FHWA personnel. In particular, the interest and assistance of Gary Evink, Larry Barfield, and Win Lindeman from FDOT and Byron Lord from FHWA were very much appreciated.

REFERENCES

1. T. Wang, D.E. Spyridakis, B.W. Mar, and R.R. Horner. Transport Deposition and Control of Heavy Metals in Highway Runoff. Report 10. Department

of Civil Engineering, University of Washington, Seattle, 1982.
2. Methods of Chemical Analysis of Water and Wastes. Report No. EPA-600/4-79-020. U.S. Environmental Protection Agency, March 1979.
3. Y.A. Yousef, M.P. Wanielista, H.H. Harper, D.B. Pearce, and R. Tolbert. Best Management Practices--Removal of Highway Contaminants by Roadside Swales. Florida Department of Transportation, Tallahassee, March 1985.
4. M.P. Wanielista, Y.A. Yousef, and H.H. Harper. Hydrology/Hydraulics of Swales. Presented at the Workshop on Open Channels and Culvert Hydraulics, Orlando, Fla., Oct. 22-23, 1983.